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CONTENTS

October, 1958

Papers

Paper 1812 October, 1958 SM 4

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

ANALYSIS FOR PARTIALLY PENETRATING SAND DRAINS

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and W. C. Boyer,[3] A. M. ASCE
(Proc. Paper 1812)

SYNOPSIS

A numerical procedure of analysis for partially penetrating sand drains has been developed and applied to an illustrative problem. Possible simplified methods of analysis are indicated and laboratory tests have been conducted to correlate these methods with observed rates of consolidation.

INTRODUCTION

The differential equation, in cylindrical coordinates, applicable to the vertical sand drain problem, is:

$$\frac{\partial u}{\partial t} = C_z \frac{\partial^2 u}{\partial z^2} + C_r \left[\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right] \tag{1}$$

where

c_z = coefficient of vertical consolidation due to vertical flow of water

c_r = coefficient of vertical consolidation due to radial flow of water

u = hydrostatic excess pore pressure

r = radial coordinate

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z = vertical coordinate

t = time

The development of this equation has been reviewed by Barron.[3] This equation combines the one-dimensional vertical flow solution as defined by Terzaghi[14] and the radial flow component solution as developed by Rendulic.[11] Carrillo[4] has shown that the above equation can be solved as two separate parts and ultimately recombined to give the complete solution.

The above solution is applicable to the condition of drains completely penetrating the compressible muck. In some instances deposits of muck are so deep that it is not economically feasible to penetrate the full depth. When such conditions arise, the solution of the flow equation will be altered by new boundary conditions and the above described solution will not be valid. The permeability of the sand drain is very high compared to the material which must be drained. Due to this high permeability, the excess pore pressure will be dissipated immediately by the free flow of water through the drain. Hence, for practical purposes, the excess pore pressure equals zero at every point within the sand pile and in particular along the outside radius of the pile, at $r = a$. Therefore, in the case of the completely penetrating sand drain, the condition:

$$u = o \text{ at } r = a, \text{ for any } t,$$

holds throughout the depth of the stratum to be drained. The boundary condition aids materially in the relative simplicity of the solution of the problem. In the case of the partially penetrating drain, however, this condition does not exist at depths greater than the depth of the pile. Therefore the previous solutions do not hold. To the knowledge of the writers, there has not been presented an analytical solution for the problem of partially penetrating drains. The numerical solution which will be presented here can be applied to all problems regardless of drainage conditions.

The Numerical Solution

Substituting $r = \sqrt{\frac{c_r}{c_z}} \cdot \bar{r}$ into Eq. (1) gives

$$\frac{\partial u}{\partial t} = C_z \frac{\partial^2 u}{\partial z^2} + C_z \left[\frac{\partial^2 u}{\partial \bar{r}^2} + \frac{1}{\bar{r}} \frac{\partial u}{\partial \bar{r}} \right] \tag{2}$$

By taking finite differences of time and distance to replace the differentials dt, $d\bar{r}$, and dz in Eq. (2), a difference equation can be developed to provide an approximate solution for this equation.

Due to the symmetry of the problem, a vertical section (Fig. 1) through the $\bar{r}$ - z plane will show completely the variation of the excess hydrostatic pore pressures. To represent the differentials of $\bar{r}$ and z by finite differences, the section is divided into a grid with finite intervals of space separating the points of the grid. Finite differences are applied to Eq. (2) at point, O, in the grid with λ_z as the finite vertical difference, $\lambda_{\bar{r}}$ as the finite radial difference and δt as the finite time interval.

Expanding the pressures in a Taylor series with respect to time at point O, and with respect to distance at points 1, 2, 3 and 4, will lead to the difference approximation:

$$u_0(t+\delta t) = \frac{c_z \delta t}{\lambda_{\bar r}^2}(u_3+u_1) + \frac{c_z \delta t}{\lambda_z^2}(u_2+u_4) + \frac{c_z \delta t}{2\bar r_0 \lambda_{\bar r}}(u_3-u_1) + \left(1 - \frac{2c_z \delta t}{\lambda_{\bar r}^2} - \frac{2c_z \delta t}{\lambda_z^2}\right)u_0 \tag{3}$$

In Eq. (3) δ t is assumed to be of the magnitude $\lambda_{\bar r}^2$ and λ_z^2 and all terms of the magnitude λ^4 and greater are eliminated. This is the most general expression of the difference equation. Each value can be chosen arbitrarily to give the degree of accuracy desired. Thus, application of Eq. (3) at point 0 would give the excess hydrostatic pore pressure at 0, for time t + δt.

Selecting a square grid such that $\lambda_z = \lambda_{\bar r} = \lambda$, we obtain:

$$u_0(t+\delta t) = \frac{c_z \delta t}{\lambda^2}(u_1+u_2+u_3+u_4) + \frac{c_z \delta t}{2\bar r_0 \lambda}(u_3-u_1) + \left(1 - \frac{4c_z \delta t}{\lambda^2}\right)u_0 \tag{4}$$

It is possible to simplify further this expression by letting $A = \frac{1}{4} = \frac{c_z \delta t}{\lambda^2}$, where A must be less than 0.5 in order to get convergence of the difference Eq. (9). This eliminates the u_0 term in the expression, thus yielding:

$$u_0(t+\delta t) = \frac{1}{4}(u_1+u_2+u_3+u_4) + \frac{\lambda}{8\bar r_0}(u_3-u_1) \tag{5}$$

The equation reduces to two simple operations: (a) the average of the pressures at the four points surrounding point, 0; plus (b) the multiple of a simple function of $\bar r_0$ times the pressure difference between the points 1 and 3 on the radial axis.

To apply the relaxation method, a sector is taken as in Fig. 1, bisected by the $\bar r$ - z plane through the volume affected by the sand pile. This plane is divided by a square grid array. The time interval δt is dictated by the choice of grid. The initial pressure is placed at each grid point in the plane. Any initial pressure distribution may be used, depending on the problems to be solved, without adding complication to the method. By applying Eq. (5) to each grid point successively, the hydrostatic excess pressure at time, $t_0 + \delta t$, is determined. Repeated application of the equation may be continued for as many intervals as are desirable.

Illustrative Problem

In order to study the accuracy of the method outlined above, a problem was worked with the sand pile completely penetrating the compressible layer. Hence three solutions could be made for comparison:

(a) Calculations of the pressures by the relaxation equation, with per cent consolidation computed by the averaging process.
(b) Calculations of the pressures by the equations of Rendulic and Terzaghi, with per cent consolitation computed by the averaging process.
(c) Calculations of per cent consolidation directly by the equations of Rendulic and Terzaghi.

The physical properties of the problem were:

18 in. drain diameter, a = 0.75 ft

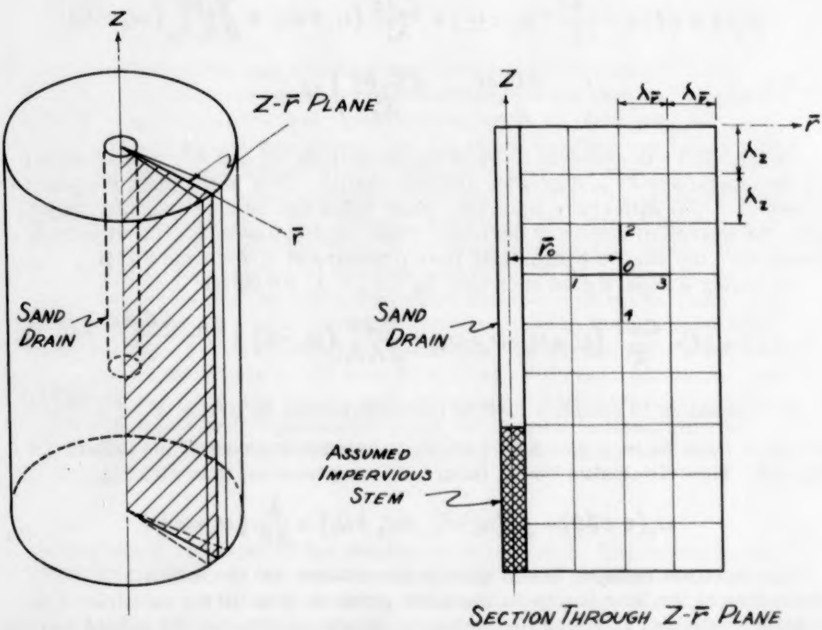


FIG. 1

GRID SYSTEM FOR PARTIALLY PENETRATING SAND DRAINS

17 ft spacing, $b = \frac{1.05 \times 17}{2} = 8.93$ ft

$n = b/a = 11.9$

$H = 16.36$ ft

$c_z = c_r = 10 \times 10^{-4}$ cm^2 per sec $= 9.3 \times 10^2$ ft^2 per day

$\lambda = \frac{(8.93 - 0.75)}{4} = 2.04$ ft

$\delta t = \frac{(2.04)^2}{4 \times 9.3 \times 10^{-2}} = 11.24$ days

$u_1 = 100$ psi (assumed constant throughout the soil layer)

The cross section, for the cylinder of soil drained, was divided into a grid system according to the above parameters. Initially, u_1, the initial excess pressure was placed at all of the interior points and the excess pressure equal to zero was placed at the drainage surfaces. The surface at $z = H$ is an impermeable surface ($\frac{\partial u}{\partial z} = 0$) and u_4 equals u_2 at all points along this surface. The surface at $r = b$ is the dividing surface between two piles ($\frac{\partial u}{\partial r} = 0$) and u_3 equals u_1 at all points along this surface.

It was decided to compute the per cent consolidation after 8, 16, 24, and 32 time intervals. For each of these times the cross-section was drawn to scale, the value of the pressure placed at each grid point, and lines of equal pressure drawn. An example of these equi-potential lines is shown in Fig. 2.

Fig. 3 illustrates the process which was used to obtain the average pressures. The horizontal grid lines divide the cylindrical volume into eight equal discs which are treated separately. On a horizontal plane at the center of the disc, the average pressure is taken between two equal pressure lines intersecting the plane, and multiplied by the area of the annular section cut by the two pressure lines. This is done in increments for the whole area. These values are summed and divided by the area of the horizontal plane giving the average pressure of the discs. An arithmetic mean of the eight discs is taken as the average pressure of the entire layer. The average per cent consolidation is expressed as:

$$\bar{U}\% = 100(u_1 - \bar{u}) = (100 - \bar{u}\%) \tag{6}$$

where $\bar{u}$ is the average excess hydrostatic pore pressure.

The values of the per cent consolidation calculated in this manner are:

Time Interval	Time in Days	Per Cent Consolidation
8	89.98	34.6
16	179.84	48.3
24	269.76	58.9
32	359.68	67.4

For the second solution, it was necessary to determine the pressures due to both vertical and radial flow and to combine them (4) as:

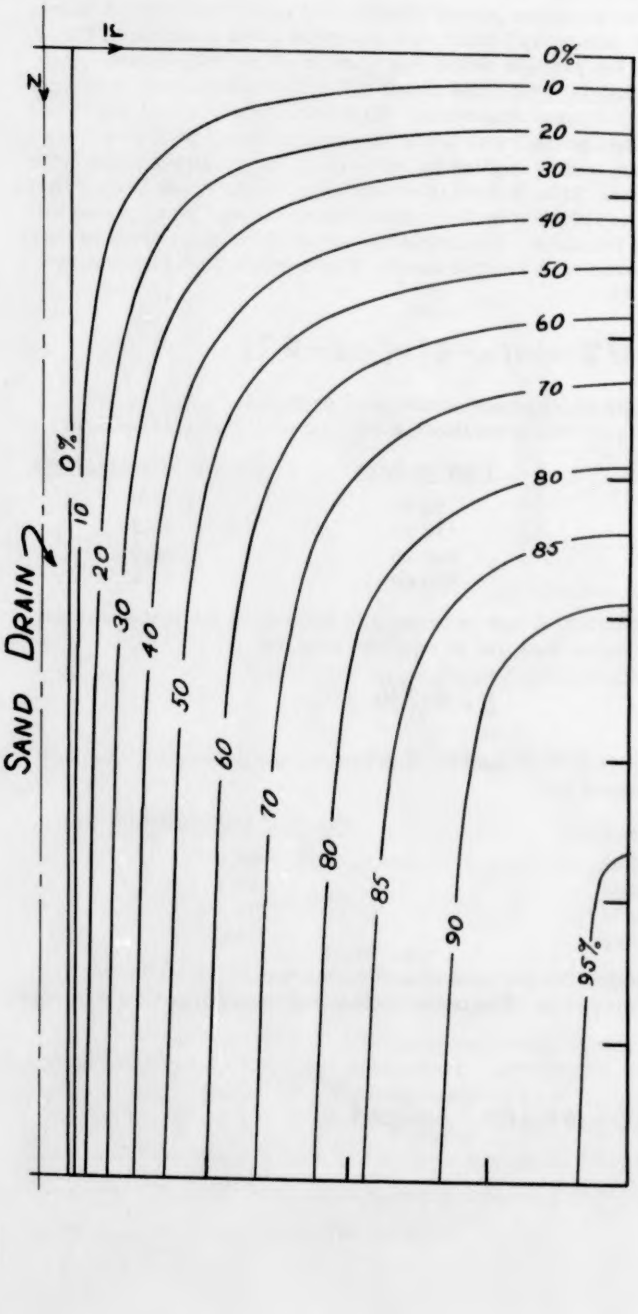
$$\bar{u} = \frac{\bar{u}_r \times \bar{u}_z}{u_1} \tag{7}$$

Using the previously described method of averaging the pressures, the percentages of consolidation are:

Time in Days	Per Cent Consolidation
89.92	32.9
179.84	47.1
269.76	56.3
359.68	64.6

For the third solution the per cent consolidation was calculated directly from the analytical formulae. The consolidation due to vertical flow was computed from:

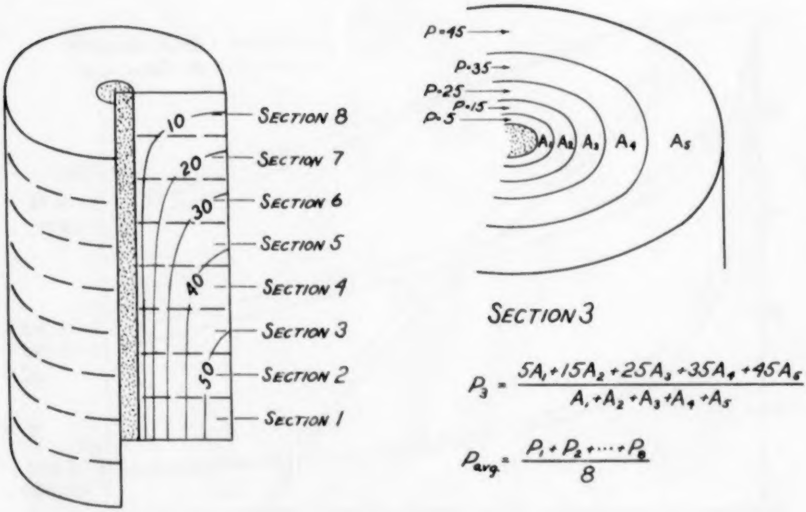
$$\bar{U}_z\% = 100\left[1 - \sum_{n=1,3,5,\ldots}^{\infty} \frac{8}{n^2\pi^2}\, \epsilon^{-\frac{\pi^2 n^2 T_z}{4}}\right] \tag{8}$$



Equi-Potential Lines Expressed As Percentage Of u,

Fig.2

Pressure Distribution At t = 89.92



$$P_3 = \frac{5A_1 + 15A_2 + 25A_3 + 35A_4 + 45A_5}{A_1 + A_2 + A_3 + A_4 + A_5}$$

$$P_{avg.} = \frac{P_1 + P_2 + \cdots + P_8}{8}$$

FIG. 3
EXAMPLE OF THE AVERAGING OF PRESSURES

The consolidation due to radial flow is computed from (2):

$$\bar{U}_r\% = 100\,\frac{\left[1 - \sum_{n=1}^{\infty} \frac{2a}{K_n} A_n z_1(K_n a)\,\epsilon^{-4b^2 n^2 T_r}\right]}{b^2 - a^2} \tag{9}$$

Having $\bar{U}_z\%$ and $\bar{U}_r\%$ the total per cent consolidation may be obtained using the equation:

$$(100 - \bar{U}\%) = (100 - \bar{U}_r\%)(100 - \bar{U}_z\%)\frac{1}{100} \tag{10}$$

The per cent consolidations so calculated are:

Time in Days	Per Cent Consolidation
89.92	33.0
179.84	46.4
269.76	56.5
359.67	66.0

By this procedure, the accuracy of the relaxation process and averaging process could be individually checked against the analytical solution. Fig. 4 shows the plots of the pressure values obtained from methods (a) and (b) for certain designated points in the cross-section. A comparison of average consolidations with time for the three methods of computation is shown in Fig. 5. It may be noted that the complete analytical solution and the analytical

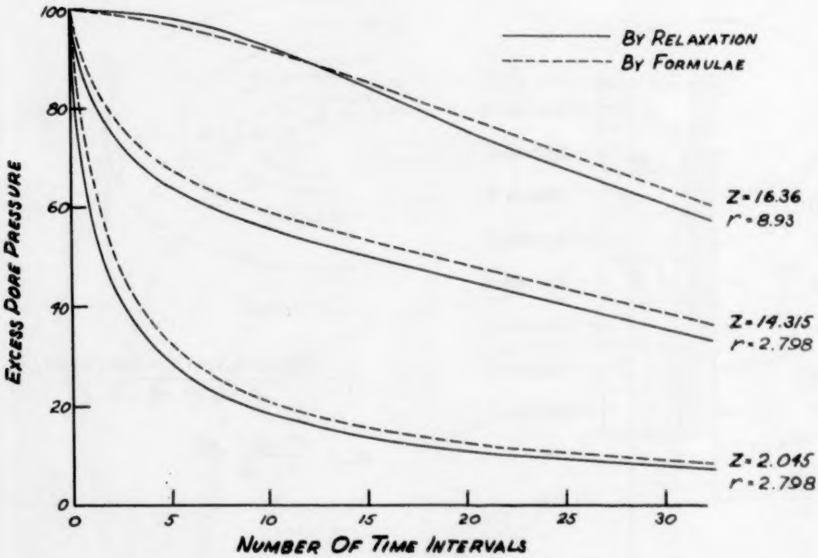


FIG. 4

COMPARISON OF PRESSURES

pressure solution with averaging nearly coincide and show no systematic error for the averaging process. The relaxation technique yields answers consistently higher by about 1 to 2 per cent consolidation. This demonstrates that the method in its entirety is sufficiently accurate to apply to practical problems.

Having demonstrated the accuracy of the method, it was applied to the problem of partially penetrating drains in order to study the effects of penetration depth on the rate of consolidation. To avoid irregularities in the grid system, and special boundary forms of the difference equation, it was assumed that the small column of earth from the bottom of the pile to the bottom of the compressible layer was impervious material.

The average per cent consolidation was computed for percentages of penetration equal to 0%, 12.5%, 50%, 75%, 87.5%, and 100% for times approximately equal to 3, 6, 9, and 12 months, respectively. The results of this study are shown in Fig. 6.

The data of Fig. 6 represent the solution for one sand pile layout. It would be presumptuous to make broad generalizations on the basis of this work. The curves do, however, show a trend which the authors believe will persist if other layouts are studied for the effects of partial penetration. The sections of the drain nearest the surface and nearest the bottom of the layer contribute least to the consolidation. If a drain is extended only a short distance into the layer it will barely affect the consolidation, but if a drain is extended nearly to the bottom of the layer it will give more consolidation per foot of drain than a completely penetrating drain.

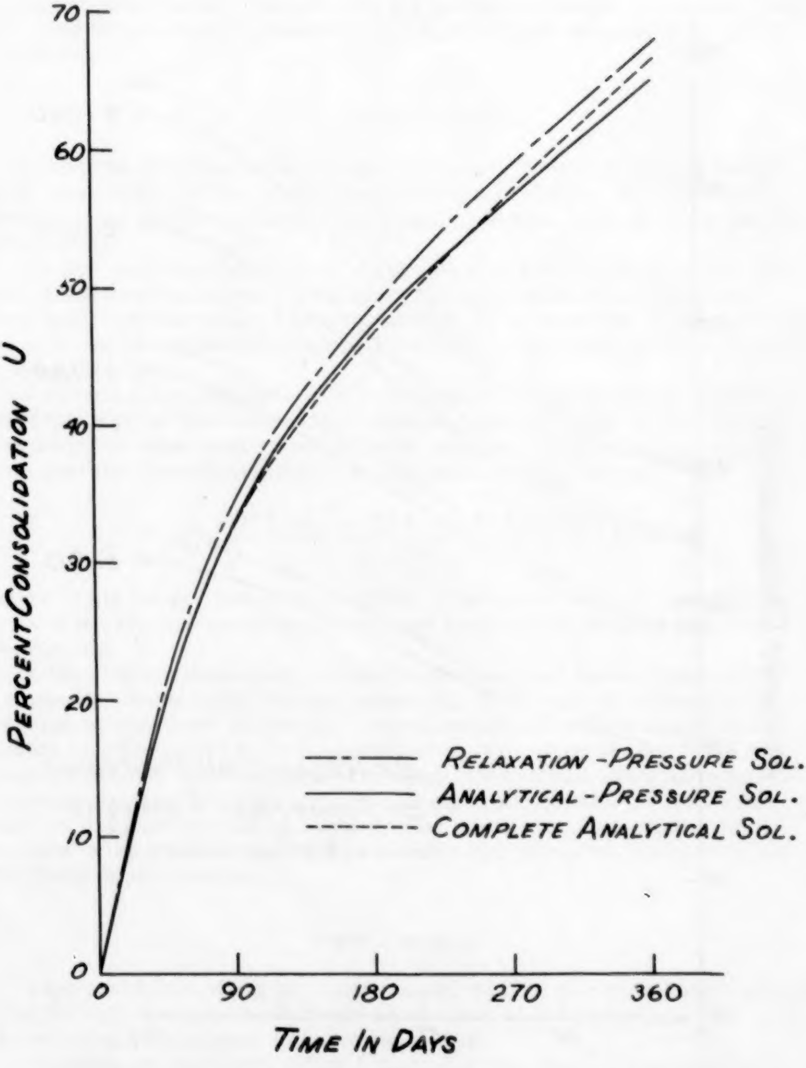


Fig. 5
Comparison Of Total Consolidations

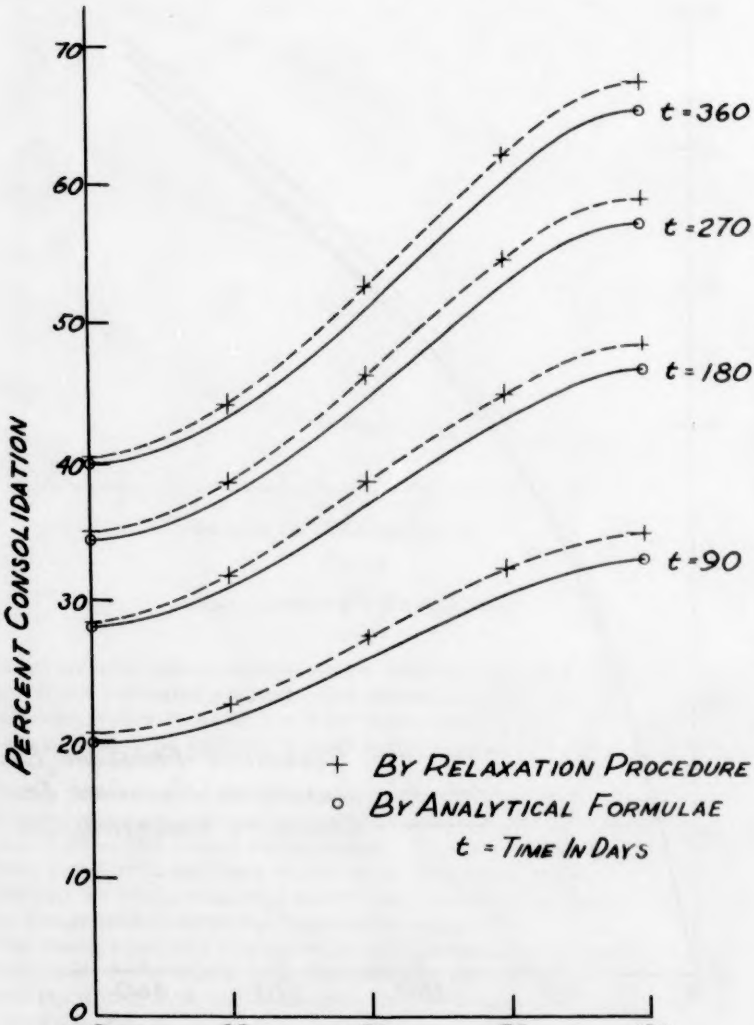


Fig. 6
The Effect Of Pile Penetration

This indicates that partially penetrating drains could be of economic advantage in some cases. However, for any particular design, a complete analysis would be necessary to determine the feasibility of using partially penetrating drains.

Proposed Approximation

It would be desirable to have a shorter method for approximating the per cent consolidation of the partially penetrating sand drains. For discussion purposes, call the horizontal plane through the bottom of the pile, the plane of penetration.

The per cent consolidation due to vertical flow and that due to radial flow can be obtained immediately from charts or calculated with relative ease from analytical formulae. The consolidation due to combined radial and vertical flow can be obtained by combining the vertical and radial effects according to Eq. (10).

A rational approximation would be to consider all of the volume above the penetration plane consolidated by combined drainage and all of the volume below the penetration plane consolidated by vertical drainage only. Thus, the total average consolidation can be approximately expressed as

$$\bar{U} = \frac{(P\%)\times(\bar{U}_{rz}\%) + (1 - P\%)\times(\bar{U}_z\%)}{100} \tag{11}$$

where P% is the per cent of penetration. This results in a straight line variation of the average consolidation between zero and one hundred per cent penetration.

In the problem illustrated, the low penetrations had consolidation values greater than the straight line approximation. In the vicinity of the fifty per cent penetration point the straight line approximation and the analytical procedure give the same per cent consolidation. Therefore, the following approximation may be considered. For small penetrations, the volume consolidated by combined drainage could be increased and the volume consolidated by vertical drainage decreased. Until further information is available, the authors' proposal for a quick but reasonable approximation would be to use the linear approximation.

Experimentation

Experimentation, using laboratory model studies, was conducted to investigate the correlation between observed values of consolidation and those predicted using the proposed approximate method.

Previous research by Kondner[8] indicated that small-scale model studies could be satisfactorily conducted for both floating embankment and vertical sand drain studies. Thus, tests were conducted on a silty swamp muck using a standard California Bearing Ratio Mold having an internal diameter of six inches and a height, including a collar extension, of eight inches. All joints were made watertight. The swamp muck was thoroughly mixed to a uniform consistency and carefully hand placed to the desired depth, sand drain inserted by the displacement method, porous glass cloth applied, and three-fourths of an inch of well graded sand was applied as a drainage medium. A consolidation pressure of 0.3 kg. per in.^{2} was applied to a bearing plate using a

modification of the standard consolidation test loading frame. The apparatus is shown in Fig. 7. Experiments were conducted for sand drain penetrations of 100%, 71.5%, 42.9%, 21.4%, and 0%.

Because of difficulty in duplicating the condition of the muck for the various tests, the standard double drainage, laboratory, consolidation tests, using a sample 2.5 inches in diameter and one inch thick, were used as a control reference. Although the consolidation characteristics varied slightly for the various tests, comparisons were made by projecting to a set of standard characteristics.

The theoretical analyses of the various tests were based on the data obtained from the standard, double drainage consolidation tests. Consolidation tests were conducted for a range of pressures including the surcharge pressure and were analyzed by the conventional square root of time fitting method.

Typical test conditions were:

G (specific gravity of solids) = 2.55

ω(moisture content) = 93.80%

e_1 (initial void ratio) = 2.39

$c_r = c_z = 16.9 \times 10^{-4}$ cm^2 per sec

$a = 0.3$ inches = radius of drain

$b = 3.0$ inches = tributary drainage radius

$N = b/a = 10$

$H = 7$ inches = depth of swamp muck

$p_z = 0.3$ kg. per cm^2 = loading intensity

$$T_z = \frac{c_z t}{H^2} = 192.5 \times 10^{-4} t \text{ (time in hrs.)}$$

$$T_r = \frac{c_r t}{4b^2} = 262 \times 10^{-4} t \text{ (time in hrs.)}$$

$\Delta H_{ult.} = 1.82$ inches

The test period was 24 hours during which readings were taken of the settlement. Theoretical determinations of the average per cent consolidation, using the linear approximation, are compared with observed values of average per cent consolidation. These results are presented in Fig. 8.

There is a remarkable correlation between theory and test which promotes confidence in the linear approximation. It is to be noted that for low depths of penetration the linear approximation gives results slightly higher than the observed values, while for medium to large depths of penetration the linear approximation gives results slightly smaller than the observed values. For complete penetration, the theoretical consolidation was slightly higher than the observed. Thus, the above results agree with those presented in Fig. 6 and seem to substantiate the accuracy of the numerical method of solution for partially penetrating sand drains.

Fig. 7. Experimental Apparatus

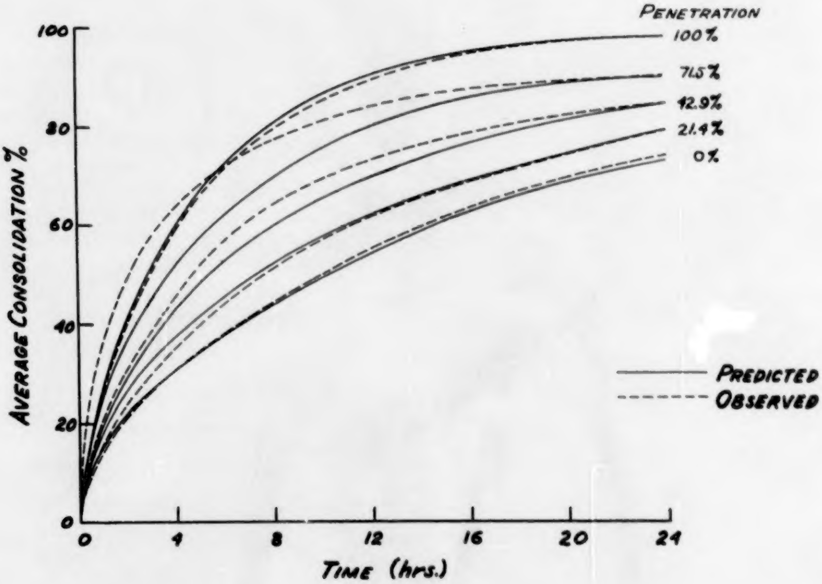


Fig. 8

Time-Consolidation Curves For Various Penetrations

CONCLUSIONS

A numerical method of analysis for partially penetrating sand drains, giving highly accurate results regardless of drainage conditions, has been developed. The linear approximation, which gives reasonably accurate results, has also been proposed.

An illustrative problem has been presented and laboratory model studies conducted to substantiate the above proposals. The sections of the drain nearest the surface and nearest the bottom of the layer contribute least to the consolidation. If a drain is extended only a short distance into the layer it will barely affect the consolidation, but if a drain is extended nearly to the bottom of the layer it will give more consolidation per foot of drain than a completely penetrating drain.

This indicates that partially penetrating drains could be of economic advantage in some cases. However, for any particular design, a complete analysis would be necessary to determine the feasibility of using partially penetrating sand drains.

ACKNOWLEDGMENT

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Paper 1813 October, 1958 SM 4

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GROUTING DEEP SOLUTION CHANNELS UNDER AN EARTH FILL DAM

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(Proc. Paper 1813)

ABSTRACT

Deep limestone solution channels caused leakage under an elliptical earth fill dam which forms a pond for log storage at Bowaters Southern Paper Corporation's newsprint mill at Calhoun, Tennessee. The foundation was stabilized by grouting and leakage was eliminated successfully.

INTRODUCTION

Bowaters Southern Paper Corporation owns and operates the largest newsprint mill in the Southern United States at Calhoun, Tennessee. The planned expansion of the plant will make it possible for the firm to maintain that position for a number of years to come.

The Charleston-Calhoun area offers many advantages for a paper mill of the size and capacity of the one built by Bowaters. The plant site is well located with respect to modes and routes of transportation because several converge and cross at the site, which is 70 miles southwest of Knoxville and 40 miles northeast of Chattanooga on U. S. Highway 11. The site is also served by the Knoxville Division of the Southern Railway System. It is at the head of navigation on the Hiwassee River, a tributary of the Tennessee River by way of which the area has access to all inland waterways of the country. Oil and gas transmission pipelines pass through the plant site and electric power is available from the TVA system. Process and cooling water is available from the Hiwassee River and the region is well adapted to producing timber for pulpwood.

Note: Discussion open until March 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1813 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 4, October, 1958.

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2. President, Schmidt Eng. Co. Inc., Chattanooga, Tenn.

Log Storage

Log storage for paper mill operation when such mills are situated in areas where logs are not floated to a plant has long been one of the problems of the paper industry. At Bowaters Southern Paper Corporation at Calhoun, Tennessee, the problem has been solved by providing a log pond capable of retaining 30,000 cords of wood. The log pond was created by dishing out the central section and constructing an earth fill dike having end radii of 245 feet at the centerline of the dike crest and with 50 feet of tangent on each side between the curves. The dike is approximately 24 feet high with water side slope 1 on 1-1/2, which is lined with 4 inches of gunnite. The dry side slope is of 1 on 3, seeded with grass. The crest width is 15 feet. A plan and sections of the log pond structure is shown in Fig. 1.

At the center of the pond a tower has been erected to house conveyor apparatus for logs moving to and from the log pond and for the pivotal points of two gantry cranes equipped with log grabs which travel along the gantry bridges that extend from the pivots to the crest of the fill where the other end of each gantry runs on self-propelled trucks on railroad rails.

The inside of the pond is 40 feet deep from the crest of the dike to the bottom of the dished portion and water is carried to a depth of 39 feet or within 1 foot of the crest of the dike. When at this depth the water surface area is 4.55 acres.

The principal advantage of this type of wood storage is the prevention of rot because wood does not deteriorate under water. Other advantages are the minimizing of fire hazard for such storage and the availability of a sizeable stockpile to equalize deliveries and log use. Deliveries are by barge, rail and truck from which all logs go through a debarking process. Then they are diverted either to the chippers or to a conveyor to the log pond, from which they can also be returned to the chippers by a reverse conveyor at will.

Geology of Area and Site

The rocks of the region are deformed Paleozoic sediments of the Ridge and Valley Province. Most of the plant is built on an outcrop belt of the Knox, and where minor structures were placed on older or younger formations the geology had little, if any, influence on the design or construction of the plant. Knox beds beneath the plant which have been encountered in core borings in the foundation under the larger buildings and log pond belong to the Upper or Ordovician part, but older Copper Ridge rocks occur to the northwest under the waste disposal ponds. Almost all of the rock under the plant is a light to medium gray, generally thick-bedded, medium crystalline to dense dolomite. The highest beds penetrated along the southeast side of the plant contained many members of dark to medium gray, fine-grained, banded limestone. A few beds of sandstone are present and in the cores from bedrock, light and dark chert is common but not abundant. Dolomite breccia is also very common, some of which is recemented and some of which is not.

The principal structural features of the area are the Knoxville Fault along which the Conasauga is thrust over the upper Knox Mascot and Kingsport beds and the Saltville Thrust with Rome and Lower Conasauga displaced above the Upper Conasauga formation. High angle cross faults also occur frequently and the present course of the Hiwassee River for several miles northwest of Charleston and Calhoun is aligned along one of these. The strike of the bedding is about north 25° east but there is a slight eastward warp in this at the

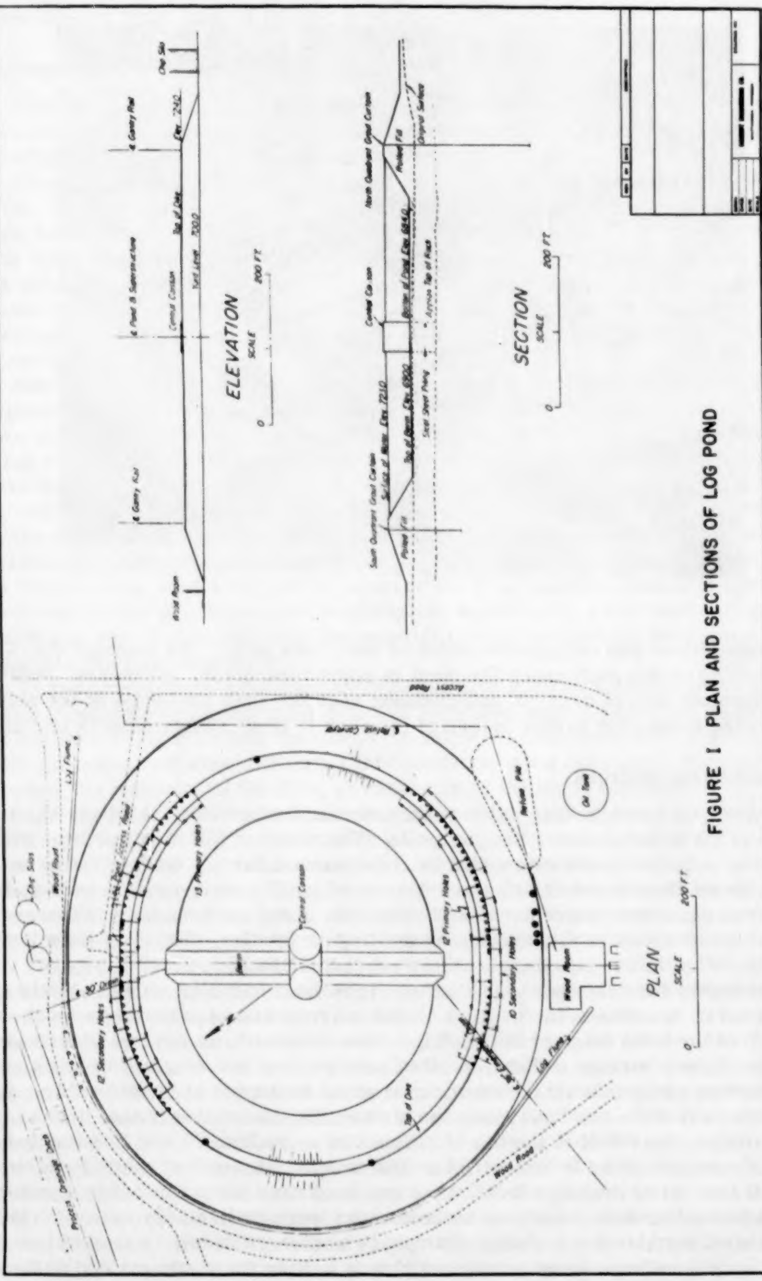

FIGURE I PLAN AND SECTIONS OF LOG POND

Bowaters Southern Paper Corp. Plant
Calhoun, Tenn. - Log Pond in Foreground

Hiwassee River and on opposite sides of the cross fault. The average dip of the bedding in the rock under the plant is approximately 40° southeast. However steeper dips of near 75° are common near the fault southeast of the plant. Due to the extremely brittle nature of the rock it is all rather closely jointed.

Solution in the Bedrock

Solution of its carbonate parts is the only form of weathering of any significance in the bedrock under the plant site. The mass of the rock has been affected by solution to the extent that is about normal for the Knox of the area. Small or medium sized cavities are developed locally along certain beds and there are numerous depressions and pinnacles in the rock surface. Where there is not excessive disturbance by faulting or jointing, the relief from the irregularities in the top of rock rarely exceeds 15 to 20 feet.

The highly fractured wall rock of the cross fault has been so extensively dissolved by cavities in the bedrock in some areas that apparently as much as 25% of the total mass of the rock has been removed. In several places locations of core borings and driven piles indicate that low areas of bedrock and cavities along this structure occur at great depth and at considerable distance away from the fault along the strike. On the southeast side of the mill building the relief in the top of bedrock is more than 70 feet and the deep channels responsible for this relief extend to a known depth of slightly more than 80 feet below drainage level. The extent of cavities in the bedrock under this dissolved area is not known since borings were drilled only to such depths where rock suitable for foundation purposes was encountered. Areas of low top of rock have been found in several places across the northeast end of the mill building and powerhouse.

As later discovered the most striking area of dissolved rock is located along the northeast side of the log pond structure.

Leakage and Cavitation Under the Log Pond

After the log pond dike had been constructed and filled with water, considerable leakage from the pond was noticed in the system of underdrains and an investigation of the cause of this leakage was initiated.

The leakage problem in the log pond dates back to the first filling in May 1954. At that time there were only a few scattered shallow borings and a few deep piles serving as spot locations for the top of bedrock. It was apparent that there was extensive solution in the bedrock under the pond. The fact that the pond was built over a group of beds in the Knox dolomite which are sometimes locally dissolved led to the conclusion that unless some remedial treatment of the bedrock was done cave-ins in the unconsolidated overburden would occur.

Since the site of the log pond was on a geologically questionable foundation exploration holes ranging in depth from 75.5 feet to 214.6 feet were spotted in strategic locations to determine the potential zones of leakage in the underlying rock if any. This preliminary exploratory drilling consisted of diamond core boring 3 inch diameter holes in which certain dye tests were made, all of which proved negative insofar as leakage from the log pond was concerned in the north quadrant of the dike. In the south quadrant however, there were evidences of foundation leakage to the site of a proposed new pumping station by virtue of log pond water rising up over the drill casings when this latter area was explored. Also certain geological weaknesses were indicated in this south quadrant which indicated the possibility of some leakage between the log pond and the Hiwassee River.

These preliminary explorations revealed areas under both the north and south quadrants of the log pond dike that indicated such a degree of cavitation in the underlying rock that the structure might be of questionable stability and that immediate repairs to correct this condition were necessary in order to protect the integrity of the dike, particularly in the north quadrant.

A concrete slab was placed in the bottom of the pond in the summer of 1954, and the interlocks of the tower piling supports were grouted as an immediate expedient to provide temporary protection until permanent corrective measures could be taken. The principal reason for this work was to secure the immediate use of the pond. The pond continued to leak seriously and in late December 1954 a cavein occurred on the south side near the wood room.

The preliminary explorations of January and February 1955 indicated an extensive system of interconnecting cavities in the bedrock, most of which were open and contained no clay or chert filling. This confirmed the earlier conclusion that the real problem was to prevent cave-in of the overburden and subsequent cave-ins in the dike. Leakage, while a nuisance, was really a secondary problem. The most seriously dissolved zone being under the north quadrant, it was reasoned that this area was the most hazardous from the standpoint of likely collapse. Drilling data indicated that solution apparently had progressed downward along certain badly fractured beds of dolomite. Solution also had spread outward from the fault along these same beds and with the removal of firm support for the overlying rock, collapsing had occurred which produced a group of open strike joints and open bedding planes causing cavities to be developed along these fractures. The deepest cavity

encountered in this area was at elevation 493 or more than 175 feet below the bed of the nearby Hiwassee River. At the center of the log pond bedrock is dissolved to some extent but on the southwest side of the pond the deepest cavities were found to be above elevation 625 or only about 50 feet below the bed of the Hiwassee River.

While the north quadrant of the log pond showed the deepest and most extensive cavitation and the south quadrant showed most potential leakage and shallower cavitation, the exploratory drilling in the east and west quadrants indicated reasonably stable foundation rock. Foundation treatment plans were consequently limited to the bad zones in the north and south quadrants.

Grouting the Foundation Rock in the North Quadrant

Drilling and grouting was started on the north side just as soon as the equipment and personnel could be assembled. First stage grout holes were core drilled 3 inches in diameter and 40 feet on centers along the crest of the dike. It was recognized that the cavitation in the north quadrant would require very large volumes of grout, much more than could be placed economically with cement or asphalt. To meet this contingency it was felt that a clay-cement mix would suffice.

Clay beds with high proportions of silt and fine sand particles, especially suitable for this type of grouting, were located on the property and this material was hauled to a central plant site in self-loading scrapers. The material consisted of an average of 44% clay, 30% silt and 26% sand of 0.4 m.m. maximum grain size. The loose weight of the soil averaged 73 pounds per cubic foot. At the plant it was stockpiled, hand screened and fed to a one-half cubic yard concrete mixer where a clay slurry was mixed. From this mixer the slurry was fed to another mixing tank where cement was added. The combined mix then passed to an agitator tank from which it was pumped to the grout holes by air-operated, piston type, slush-fitted grout pumps.

Water-cement ratios were varied to suit grouting conditions at each hole. In general the grout was kept as stiff as the pump could handle. No viscosity tests were run. The grout generally was the consistency of a thick cream. Test mixes up to 1 part cement and 7 parts clay were prepared and the final mix selected consisted of 1 part cement to 5 parts of clay by volume.

Because of the extensive cavitation it was virtually impossible to set grout packers for stage grouting and this was given up early in the interests of filling the voids by gravity type grouting. Furthermore it was recognized that while the consolidation of the rock under the fill structure was the required objective, the extent of the cavitation was so great that the application of more than minimum pressure would have caused the grout to move unreasonably far laterally, resulting in unnecessary expenditures.

Accordingly open end pipes were inserted to the bottoms of the holes and grout was pumped into the hole for one 8 hour shift at a time. The amount of grout going into the hole was regulated by a header and a return grout line in order to avoid spilling over the top of the hole casing and wasting grout. After the grout thus placed had hardened, the hole was sounded and the grout pipe reinserted to the top of the hardened grout, after which the process was repeated until the hole finally filled to the top. Where large amounts of grout were taken the hole was generally reamed and regrouted to check the effectiveness of the work. The same procedure was used on second and third stage holes.

In this north quadrant the depth of the 14 primary holes averaged 66.5 feet in the fill and overburden and 131 feet in the rock below the overburden. 142,358 cubic feet of grout were placed in the rock for an average of 77.6 cubic feet per foot of hole drilled in rock.

Secondary holes were spaced midway between the primary holes, bringing all primary and secondary holes to a spacing of 20 feet on centers in the north quadrant. The 12 secondary holes averaged 64 feet deep in the fill and the overburden and 119 feet deep in the underlying rock. The rock portion of these holes accepted 35,064 cubic feet of grout for an average of 24.6 cubic feet per foot of hole, showing the substantial effectiveness of the grouting in the primary holes.

Third stage holes were drilled midway between the primary and secondary holes, bringing the hole spacing to 10 feet on centers. Twenty-three of these third stage holes averaged 68.9 feet deep in overburden and fill and 96.3 feet deep in rock. These 23 holes took 44,033 cubic feet of grout in the rock portion of the hole for an average of 19.9 cubic feet per foot of hole showing the further effectiveness of the primary and secondary grouting work.

An examination of the grouting records for the first three stages indicated five places where it appeared that fourth stage holes, bringing the hole spacing to 5 feet on centers, might be required. These 5 holes averaged a depth of 70 feet in the fill and the overburden and 123.8 feet in the rock. The rock portions of these holes were grouted with 1,870 cubic feet of grout for an average of 3.0 cubic feet of grout per foot of hole showing that the area was generally tight from the grouting that had been done in the previous stages.

A section through the north quadrant foundation, showing cavitation, holes drilled and grout taken is shown in Fig. II.

Grouting the South Quadrant Foundation Rock

In the south quadrant where leakage to the Hiwassee River was suspected, clay cement grouts might have been subject to segregation and washing out of the clay so it was decided to use neat cement grout. Grout mixes were varied to suit the foundation conditions and water cement ratios varied from 1.5 to 0.6. Quick set, drop type packers were set and used in stages throughout the grouting of the south quadrant. When grout takes were high even with the 0.6 mix grout, calcium chloride was added to accelerate the setting time and prevent the grout from washing away and to seal channels where water leakage was taking place. After grout had set, the holes were reamed and regrouted.

Three inch diameter primary, secondary and third stage holes were core drilled on 40, 20 and 10 foot centers respectively. Twelve primary holes averaged 61.4 feet deep in the fill and overburden and 64.0 feet deep in the rock. The rock portions of these holes took 8 cubic feet of cement grout per linear foot of hole.

Ten secondary holes averaged 62.1 feet deep in fill and overburden and 29.6 feet deep in rock. The rock portions of these holes took 9.2 cubic feet of grout per foot of depth.

Twenty-one third stage holes averaged 62.8 feet deep in fill and overburden and 27 feet deep in rock. The rock portions of these holes took 11.9 cubic feet of grout per linear foot of hole. No fourth stage holes were required.

Although the grout takes per foot of hole did not get less for the secondary and third stage holes as would normally be expected and as revealed in the grouting of the north quadrant, the explanation for this seems to lie in the

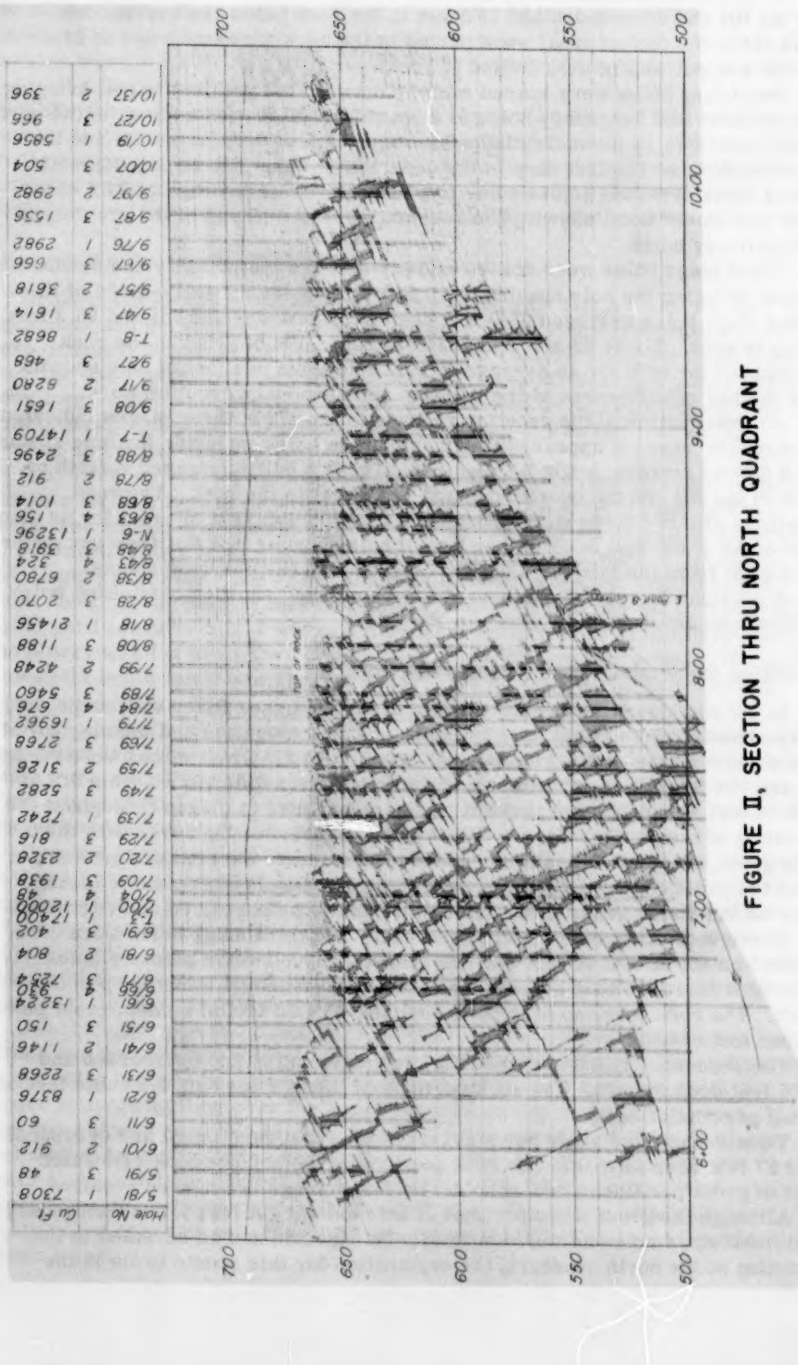

FIGURE II, SECTION THRU NORTH QUADRANT

fact that the second and third stage holes were drilled in rock comparatively short depths with respect to the primary holes. It is natural for the top portions of any holes drilled in rock to accept more grout per foot of hole than the lower portions of deeper holes. To this extent there was nothing inconsistent in this work and the character of the surface rock probably was such as to require closer spacing of primary holes or as seen after completion, that all holes on 10 foot centers in effect were of equal primary importance. The combined total take of grout per linear foot of hole drilled in rock was 9.5 cubic feet.

A section through the south quadrant showing cavitation, drilling and grout takes is shown in Fig. III.

Grouting Contact Between Overburden and Rock

After all holes in the north and south quadrants had been grouted up to the top of rock it was felt that a necessary adjunct to insuring the stability of the structure was to grout the contact between the rock and the overburden and also the overburden and fill material. Grouting this contact, overburden and fill was accomplished by keeping the drill casing filled with fluid grout while pulling it. The two operations were coordinated until the entire openings were completely filled with grout.

This procedure provided support in areas where caving of the overburden may have taken place and where same might have fallen into the rock cavities below, leaving a void in one or more places in the overburden and perhaps even in the fill. That this was a wise precaution is indicated by the fact that primary holes in the north quadrant took 577 cubic feet of grout per hole, or 8.7 cubic feet of grout per linear foot of hole drilled in the fill and the overburden. The secondary holes in the north quadrant took 39 cubic feet of grout per hole or 0.6 cubic foot of grout per linear foot of hole in the fill and overburden. Third stage holes required 21.9 cubic feet of grout per hole or 0.3 of a cubic foot per foot of hole in the fill and overburden. The fourth stage holes, of which there were only 5, took 52.8 cubic feet of grout per hole or 0.8 cubic feet per foot of hole drilled in the fill and overburden which is a departure from the trend of the primary, secondary and third stage holes, but which probably is explained by the fact that there were only 5 holes, one of which took over half of the total amount of grout thus placed thereby distorting the general relationship.

In the south quadrant no uniformity was established in grouting the contact, overburden and fill. The grout takes per hole were 14.2 cubic feet for primary holes, 18.6 cubic feet per hole for secondary holes and 34.2 cubic feet per hole for third stage holes respectively. Grout takes per foot of hole drilled in the fill and overburden were 0.2, 0.3 and 0.5 cubic feet respectively for holes spaced 40, 20 and 10 feet on centers. The explanation for this probably lies in the fact that several minor cave-ins had been experienced in the fill in the south quadrant and it is therefore quite likely in retrospect that the effective spacing of the primary holes under prevailing conditions actually should have been about 10 feet on centers instead of 40 feet on centers. The combined grout take per hole for the contact and in overburden and fill was 25 cubic feet and the combined grout take per foot of hole drilled in overburden and fill was 0.4 cubic foot. Table I is a summary of all drilling and grouting operations.

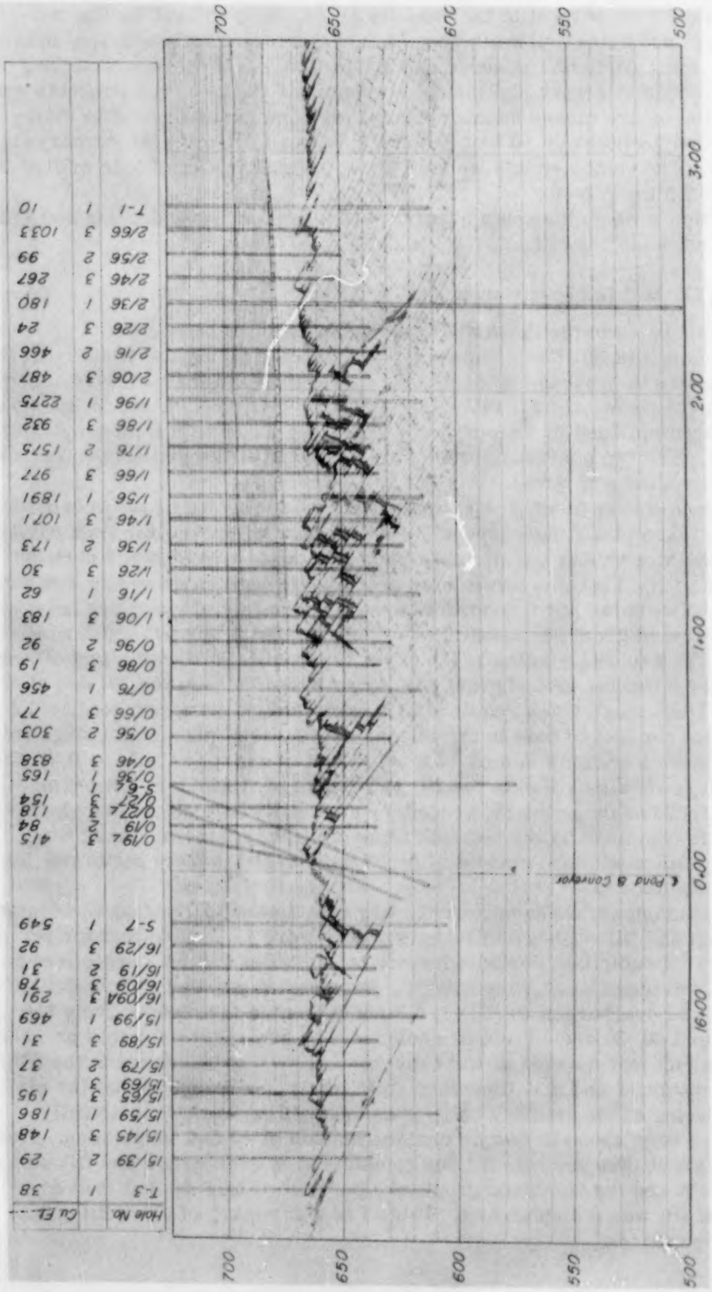

FIGURE III SECTION THRU SOUTH QUADRANT

TABLE I
DRILLING & GROUTING SUMMARY
BOWATERS LOG POND

	No. Holes	Drilling, Lin. Ft. Over-burden	Drilling, Lin. Ft. Rock	Drilling, Lin. Ft. Total	Grouting, Cu. Ft. Contact and Over-burden	Grouting, Cu. Ft. Rock	Grouting, Cu. Ft. Total	Grout Per Ft. of Hole, Cu. Ft. Over-burden	Grout Per Ft. of Hole, Cu. Ft. Rock	Grout Per Ft. of Hole, Cu. Ft. Totals	Grout Per Hole, Cu. Ft. Over-burden	Grout Per Hole, Cu. Ft. Rock	Grout Per Hole, Cu. Ft. Totals
North Quadrant													
Primary	14	931	1,834	2,765	8,076	142,358	150,434	8.7	77.6	54.4	577	10,168	10,745
Secondary	12	768	1,429	2,197	468	35,064	35,532	0.6	24.6	16.2	39	2,922	2,961
Third Stage	23	1,584	2,216	3,800	504	44,033	44,537	0.3	19.9	11.7	22	1,914	1,936
Fourth Stage	5	351	619	970	264	1,870	2,134	0.8	3.0	2.2	53	374	427
Total	54	3,634	6,098	9,732	9,312	223,325	232,637	2.6	36.6	23.9	172	4,136	4,308
South Quadrant													
Primary	12	737	768	1,505	171	6,110	6,281	0.2	8.0	4.2	14	509	523
Secondary	10	621	296	917	186	2,703	2,889	0.3	9.2	3.2	19	270	289
Third Stage	21	1,320	568	1,888	717	6,744	7,461	0.5	11.9	4.0	34	321	355
Fourth Stage	0	0	0	0	0	0	0	0	0	0	0	0	0
Total	43	2,678	1,632	4,310	1,074	15,557	16,631	0.4	9.5	3.9	25	362	387
Totals, Both Quadrants													
Primary	26	1,668	2,602	4,270	8,247	148,468	156,715	5.0	57.1	36.6	317	5,710	6,027
Secondary	22	1,389	1,725	3,114	654	37,767	38,421	0.5	21.9	12.4	30	1,717	1,747
Third Stage	44	2,904	2,784	5,688	1,221	50,777	51,998	0.4	18.2	9.2	28	1,154	1,182
Fourth Stage	5	351	619	970	264	1,870	2,134	0.8	3.0	2.2	53	374	427
Total	97	6,312	7,730	14,042	10,386	238,882	249,268	1.6	30.9	17.7	107	2,463	2,570

Results

The results of the first, second, third and fourth stage grouting programs in the north quadrant have been extremely gratifying and indicate that the badly broken foundation rock has now been stabilized to such an extent that the overlying dike is no longer in danger from possible collapse below and to this extent the expenditures made in this area have been fully justified as protection to the much greater investment in the overall log pond and its attendant equipment. Since there were no visible results from dye leakage tests in this north quadrant it seems obvious that leakage in the rock in this area was probably of a negligible quantity but served its purpose by suggesting the need for the explorations that revealed the unstable conditions in this foundation.

That the clay-cement grout hardened well in place was attested by the fact the material was capable of being cored in subsequently drilled holes after the grout had set. Numerous clay-cement cores were recovered. No compression tests were run on this grout but permeability tests indicated that 7 day old specimens would withstand 20 pounds per square inch of water pressure against a 3 inch wall of the material for an average of 20 minutes before showing any signs of leakage.

On the south side the effectiveness of the grouting was indicated significantly upon its completion when water in the drill casings at the proposed pumping station site no longer rose above the top of these casings. Also no unwatering problems were experienced in the construction of this pumping station whereas previously it had been reasonably well established that leakage from the log pond to this area was taking place in substantial quantities.

An additional measure of the effectiveness of the grouting was observed upon completion when the valve on the underdrain pipe was closed and leakage from the pond was no longer observed. It is therefore felt that in addition to correcting the leakage condition that existed, the much larger hazard that existed prior to the grouting in the form of potential settlement of the overburden into the rock openings has accomplished a protection to the investment in not only the log pond but all of the equipment required for the operation of this feature.

While it is not unusual for local areas of widespread solution to occur in the Knox there is not much detailed data available on the extent and nature of deep solution below the water table. These observations are presented as an example of just how extensive cavities have developed in one instance. In order to obtain the required subsurface information to design and build a plant it is not always necessary to drill to the bottom or over the full areal extent of all cavernous areas and for this reason many details are undeveloped as was illustrated at this location.

ACKNOWLEDGMENTS

The writers are indebted to Bowaters Southern Paper Corporation for permission to publish the work and results of this interesting problem. Specifically they are indebted to Messrs. K. L. Elderkin, President and E. L. Cowan, Chief Engineer of Bowaters Research and Development Corporation under whose general supervision the work was done and to Mr. Otha Winningham, Mechanical Superintendent who was charged with production supervision of the work. Harold Pickens was drilling and grouting superintendent for Bowaters Southern Paper Corporation and Melvin L. Downs was Resident

Engineer for the Schmidt Engineering Company which directed the engineering planning and supervision for the project. The writers were consultants to the project in the capacities of geologist and engineer. All work was done on a force account basis.

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Proceedings of the American Society of Civil Engineers

COMPUTATION OF THE STABILITY OF SLOPES

Otto H. Meyer, M. ASCE[1]
(Proc. Paper 1824)

SYNOPSIS

A problem that has confronted engineers for a great number of years is that of determining the stability of earth slopes. This problem has received a great deal of study by various researchers and a number of solutions have been offered. These solutions have been based on Coulomb's law: the unit shearing strength of the soil is the unit cohesion plus the product of the unit pressure normal to the plane of shear and the tangent of the angle of internal friction. Solutions have followed the form of establishing equilibrium equations, where the stresses result from the weight of the material lying above an assumed rupture surface and the resisting forces are those of Coulomb's law. Unfortunately these various solutions have required complex and laborious computations. There is presented here a solution expressed in simple terms, accompanied by graphs as aids to ready application.

Analysis of Forces

Various assumptions as to the shape of the rupture surface have been made. That it is curved has been shown by numerous investigations of actual slides. Among these investigations there will be mentioned only that of the Swedish Geotechnical Commission.[2] The most commonly used methods of analysis involve the assumption that the rupture arc is circular, and that assumption is adopted for this analysis.

The solution given here will be based on the further assumption that the rupture arc passes through the toe of the slope and the edge of the crown.

Note: Discussion open until March 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1824 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 4, October, 1958.

1. Chf. Bldg. Insp., Marin County, Calif.
2. Statens Järnvägars Geotekniska Kommission 1914-22; Slutbetankande, Stockholm, 1922.

As will be shown later, corrections may be applied to the solution for any other set of conditions. A prism of unit length along the face of the slope is considered.

Reference is made to the diagram of Fig. 1. There is shown a terrace of height H and inclination of slope i. A possible rupture arc, to be investigated, is shown with center at 0, central angle 2y, and radius R. The segment of earth above the arc has the area A and weight W, with its center of gravity at the point C.G. The horizontal projected distance from 0 to C.G. is d. Other quantities involved are:

w, the unit weight of the soil
c, the unit cohesion
C, the total cohesion integrated along the arc
ϕ, the angle of internal friction
P, the total normal force acting across the arc
F, the factor of safety

Taking moments about 0, the factor of safety of the slope against sliding on the arc shown is the ratio of the moment of the cohesion plus friction to the moment of the weight. In order to derive these moments we find the following:

$$A = R^2 y - R \sin y \, R \cos y = R^2(y - \sin y \cos y) \tag{1}$$

$$R = \frac{H}{2} \csc i \csc y \tag{2}$$

$$A = \frac{H^2}{4} \csc^2 i (y \csc^2 y - \cot y) \tag{3}$$

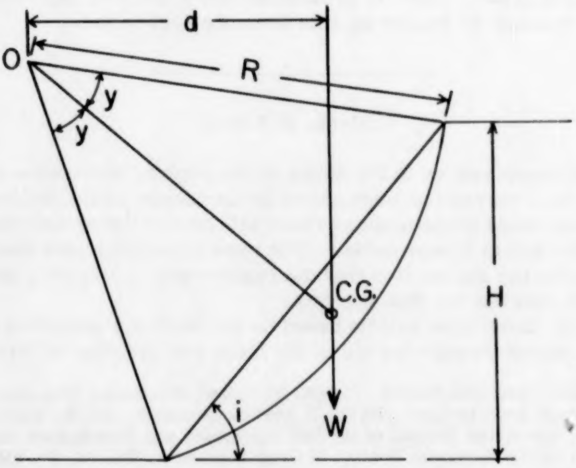

Fig. 1

Since d is sin i times the distance from 0 to C.G. (which is the cube of the chord divided by 12 times the area of the segment) it follows that:

$$d = \frac{H^3 \csc^3 i \sin i}{12\ A} = \frac{H^3 \csc^2 i}{12\ A} \tag{4}$$

The moment of the weight is thus:

$$Wd = wAd = \frac{wH^3}{12} \csc^2 i \tag{5}$$

Sliding is resisted by cohesion and friction. The moment arm of the cohesion is R and its moment is:

$$CR = 2cRy\ R = 2cR^2 y \tag{6}$$

From (2) and (6):

$$CR = \frac{cH^2}{2} y \csc^2 i \csc^2 y \tag{7}$$

The maximum friction force which may be developed is the product of the sum (not the resultant) of the normal forces, P, and tan ϕ. Its moment arm is also R and its moment is:

$$PR \tan \phi = \frac{PH}{2} \csc i \csc y \tan \phi \tag{8}$$

The factor of safety is the ratio of the moment of the cohesion plus the moment of the friction to the moment of the weight; from (5), (7), and (8), this is:

$$F = \frac{\frac{1}{2} cH^2 y \csc^2 i \csc^2 y + \frac{1}{2} PH \csc i \csc y \tan \phi}{wH^3 \csc^2 i / 12} \tag{9}$$

This is then simplified to:

$$F = \frac{6cy \csc^2 y}{wH} + \frac{6P \csc y \tan \phi}{wH^2 \csc i} \tag{10}$$

The two terms of equation (10) will now be considered individually. The expression c/wH, which is found in the first term, is dimensionless, and is seen to be independent of the location of the rupture arc. This expression will now be designated as K. The remainder of the first term is then 6y $\csc^2 y$. For convenience in plotting, the reciprocal of this expression will be taken, and will be designated as f_1. The first term of equation (10) then becomes K/f_1. The function f_1 is a function of y alone, and is shown plotted against y in Fig. 2.

In the second term of equation (10) we recognize P tan ϕ as being the total tangential friction force. The remainder of the term is the reciprocal of the tangential force tending to cause sliding, ΣT, or:

$$\Sigma T = \frac{wH^2 \sin y}{6 \sin i} \tag{11}$$

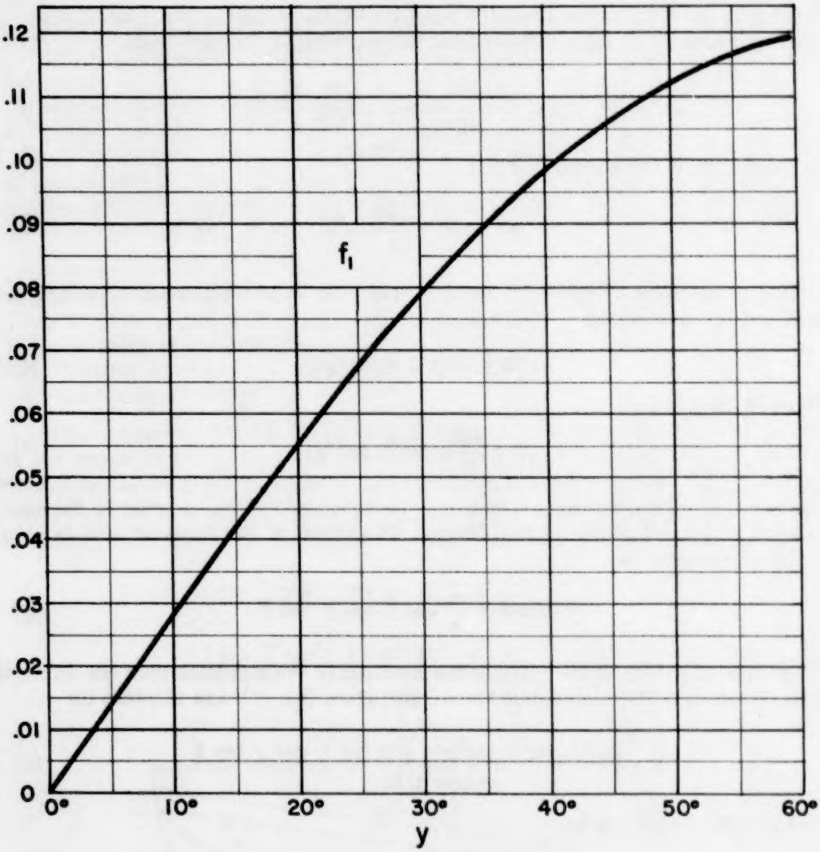

Fig. 2

We will now determine P as follows: we will divide the mass above the rupture arc into vertical slices. Each slice, as shown on Fig. 3, has a height h and produces a vertical unit load of wh. If the slice is located at a point where the angle of slope of the surface of rupture is $i + j$, the normal component of the unit load will be $wh \cos(i + j)$, and as this is spread over an arc length of $1/\cos(i + j)$, the normal unit pressure will be:

$$p = wh \cos^2(i + j) \tag{12}$$

The height h is found to be:

$$h = \frac{R (\cos j - \cos y)}{\cos i} \tag{13}$$

$$p = \frac{wR}{\cos i}(\cos j - \cos y)(\cos i \cos j - \sin i \sin j)^2 \tag{14}$$

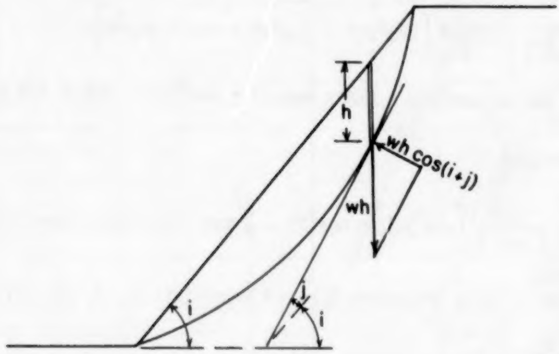

Fig. 3

The sum of all the normal forces will be:

$$P = R\int_{-y}^{y} p\,dj = \frac{wR^2}{\cos i}\int_{-y}^{y}(\cos j - \cos y)(\cos i \cos j - \sin i \sin j)^2 dj \tag{15}$$

$$P = \frac{wR^2}{\cos i}\int_{-y}^{y}(\cos^3 j \cos^2 i - 2\sin i \cos i \cos^2 j \sin j + \sin^2 i \sin^2 j \cos j$$

$$- \cos^2 i \cos y \cos^2 j + 2\sin i \cos i \cos y \sin j \cos j - \sin^2 i \cos y \sin^2 j)dj$$

$$= \frac{wR^2}{\cos i}\Big[\Big(\frac{\sin j \cos^2 j}{3} + \frac{2\sin j}{3}\Big)\cos^2 i + 2\sin i \cos i \frac{\cos^3 j}{3}$$

$$+ \sin^2 i \frac{\sin^3 j}{3} - \cos^2 i \cos y \Big(\frac{\sin j \cos j}{2} + \frac{j}{2}\Big)$$

$$+ 2\sin i \cos i \cos y \frac{\sin^2 j}{2} - \frac{\sin^2 i \cos y}{2}(j - \sin j \cos j)\Big]_{-y}^{y}$$

$$= \frac{wR^2}{\cos i}\Big[\frac{2\cos^2 i}{3}(\sin y \cos^2 y + 2\sin y) + \frac{2\sin^2 i}{3}\sin^3 y$$

$$- \cos^2 i \cos y(\sin y \cos y + y) - \sin^2 i \cos y(y - \sin y \cos y)\Big] \tag{16}$$

$$P = \frac{wR^2}{\cos i}\left\{\frac{2\sin y}{3}\left[\cos^2 i(2 + \cos^2 y) + \sin^2 i \sin^2 y\right]\right.$$
$$\left. - \cos y\left[\cos^2 i(y + \sin y \cos y) + \sin^2 i(y - \sin y \cos y)\right]\right\} \quad (17)$$

This simplifies to:

$$P = \frac{2wR^2}{3 \cos i}\left[\sin^3 y(1 + \cos^2 i) - \frac{3}{2}\cos y(y - \sin y \cos y)\right] \quad (17a)$$

As $R = \frac{H}{2} \csc i \csc y$ (Equation (2)), we substitute for R and arrive at:

$$P = \frac{wH^2}{6 \cos i \sin^2 i}\left[\sin y(1 + \cos^2 i) - \frac{3}{2}\cos y(y \csc^2 y - \cot y)\right] \quad (18)$$

Combining this with equation (11) we get for the second term of equation (10):

$$\frac{\tan \phi}{\sin i \cos i}\left[(1 + \cos^2 i) - \frac{3}{2}\cot y(y \csc^2 y - \cot y)\right]$$

which simplifies to: $\frac{\tan \phi}{\tan i} f_2$, where:

$$f_2 = 1 + \sec^2 i\left[1 - \frac{3}{2}\cot y(y \csc^2 y - \cot y)\right] \quad (19)$$

This function of y and i is plotted on Fig. 4.

The formula for the factor of safety now becomes:

$$F = \frac{K}{f_1} + f_2\frac{\tan \phi}{\tan i} \quad (20)$$

Determination of the Critical Arc

The derivation outlined above gives the safety factor for a particular arc with a certain value of y. The critical arc, giving the lowest safety factor, must be found. That is, the value of y must be found which gives the lowest safety factor, with the given values of ϕ, c, w, H, and i. This means that the derivative of P with respect to y must be zero. Then:

$$\frac{d\left(\frac{K}{f_1} + f_2\frac{\tan \phi}{\tan i}\right)}{dy} = 0 \quad (21)$$

$$K\frac{d(1/f_1)}{dy} = -\frac{\tan \phi}{\tan i}\frac{df_2}{dy} \quad (22)$$

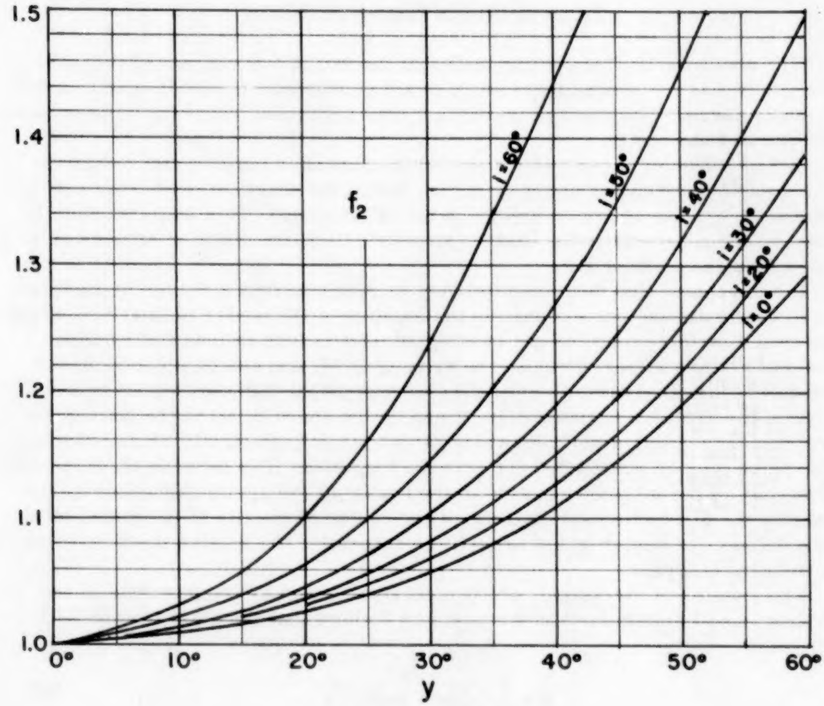

Fig. 4

$$\frac{\frac{d(1/f_1)}{dy}}{\frac{df_2}{dy}} = -\frac{\tan \phi}{K\tan i} = \frac{d(6y \csc^2 y)/dy}{\sec^2 i \; d\left[1 - \frac{3}{2}\cot y(y \csc^2 y - \cot y)\right]/dy} \tag{23}$$

$$\frac{-\tan \phi}{K \tan i} = \frac{6 \csc^2 y(1 - 2y \cot y)}{\frac{3}{2} \sec^2 i \csc^2 y(3y \csc^2 y - 3\cot y - 2y)} \tag{24}$$

$$\frac{\tan \phi}{K \sin i \cos i} = \frac{8y \cot y - 4}{3y \csc^2 y - 3\cot y - 2y} \tag{25}$$

In Fig. 5 the expression $\frac{\tan \phi}{K \sin i \cos i}$ is plotted against y. For any set of values of ϕ, c, w, H, and i the value of y for a minimum F can thus be determined without trial and error.

Effect of Shift of Center of Gravity

The above derivation presupposed that the arc of failure passed through the crown and toe of a uniform slope in a homogeneous material, which is true in many cases. The mass above the arc was a circular segment, lying between the arc and the chord.

Let us consider a case where the earth mass is of varying unit weight or has a surface extending above or falling below the chord, as where the arc passes behind the crown or below the toe of the slope. This also includes the case where a superimposed load or structure is on the slope or so near as to fall within the critical arc.

It can be seen that this situation may be resolved into a change in the total weight of the mass and a change in the location of the center of gravity. While the load distribution on the arc is affected, this change in load distribution will have only a minor effect on the value of P/W, and can be ignored unless the safety factor is close to critical. We may divide the total weight by the area of the circular segment determined by the chord between the ends of the arc, and thus obtain a modified value of unit weight, which will be called w', and which may be substituted for w in the formulae. The location of the center of gravity of the segment relative to the center of the arc is expressed by the quantity d. The horizontal distance of the center of gravity of the actual total weight from the center of the arc will be called d'. This value must be found by detailed analysis.

The moment of the weight, which must be resisted by the moments of the cohesion and friction, is thus Wd'. It now follows that the safety factor will be:

$$F = \frac{d}{d'}\left(\frac{wK}{w'f_1} + f_2\frac{\tan \phi}{\tan i}\right) \tag{26}$$

It will be noted that the method of avoiding trial and error in determining the critical value of y, by use of the curve of Fig. 5, is not applicable in this case, as K is no longer independent of y: H remains constant, but w' will have a different value for each value of y; and, where a portion of the mass becomes saturated due to seepage, or other conditions of non-homogeneity of the soil exist, the cohesion c may vary with different values of y.

For convenience in finding A, the quantity: $y \csc^2 y - \cot y = \frac{4A}{(\text{chord})^2}$ is plotted against y in Fig. 6.

The safety factor for any trial points of emergence of the arc having been determined, various other points of emergence may be tried in order to find the critical points.

Effect of Seepage

The effect of seepage, such as to produce saturation of a portion of the earth segment, may be calculated as follows:

The location of the seepage water surface must be established and plotted on a cross section of the slope, as in Fig. 7. Then, if:

c_s is the reduced value of cohesion of saturated soil

$2y'$ is the central angle of the portion of the trial arc below the seepage water surface

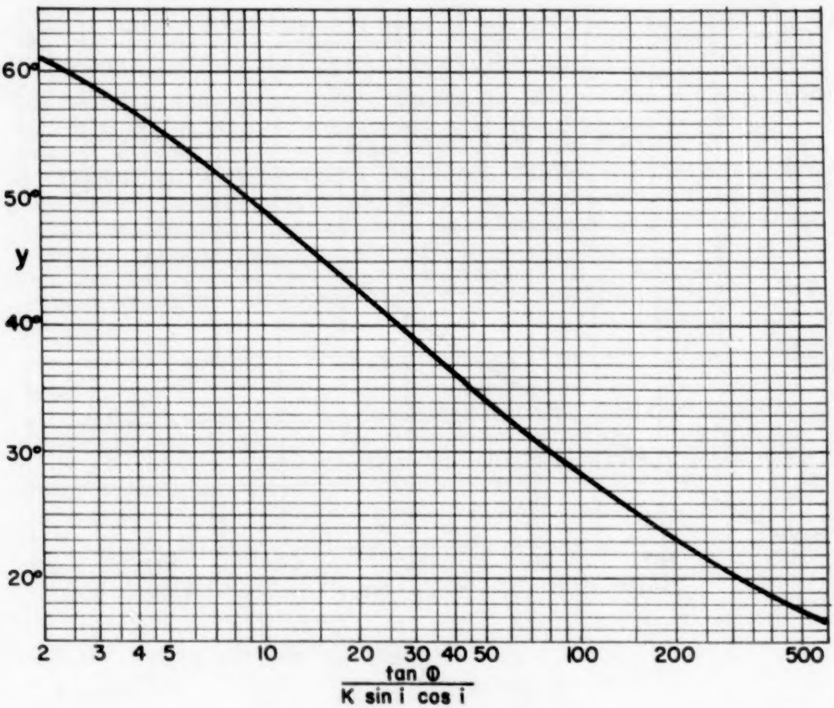

Fig. 5

c' is the weighted mean cohesion $= \dfrac{c(y - y') - c_s y'}{y}$

U is the hydrostatic uplift

the moment of the weight W is not reduced, all elements of hydrostatic uplift being radial, and, from equation (11):

$$F = \frac{c'}{wHf_1} + \frac{\tan \phi}{\tan i} f_2 - \frac{6U \tan \phi \sin i}{wH^2 \sin y} \qquad (27)$$

U is approximately equal to 62.4 times the saturated area of the segment; however this approximation may not be on the safe side. In the flow net of Fig. 7 the equipotential lines, normal to the flow lines, represent lines of equal total head. It will be noted that the hydrostatic pressure at any point is not the static head of a vertical column of water to the water surface, but that of a column whose height is the difference in altitude of the point in question and the point where the equipotential line reaches the water surface. If we add these elements of pressure along the arc, the sum (not the resultant) will be the exact value of U.

The inclination of the equipotential lines may be neglected in preliminary trials for determination of the critical value of y. Thus, in preliminary trials, the material lying below the seepage line is assumed to be buoyed up by a

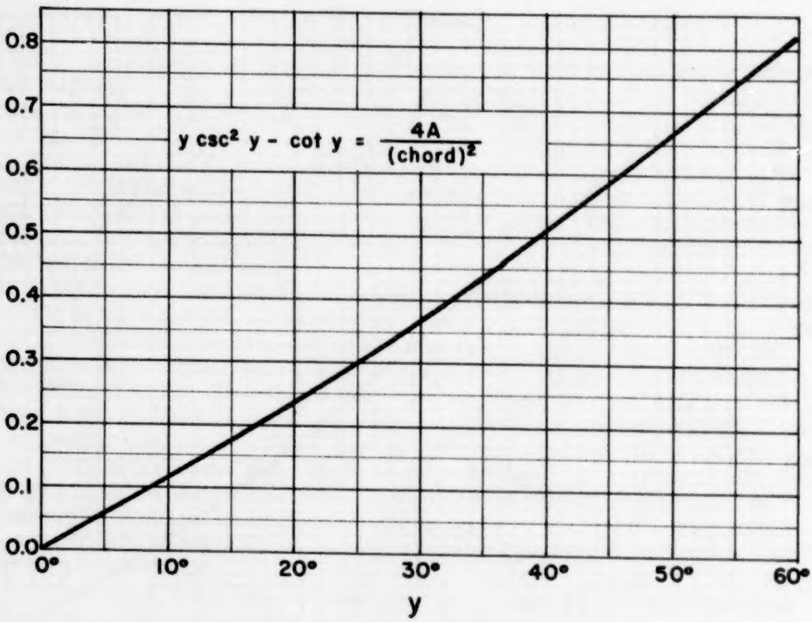
$y \csc^2 y - \cot y = \frac{4A}{(chord)^2}$
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0°
10°
20°
30°
40°
50°
60°
y

Fig. 6

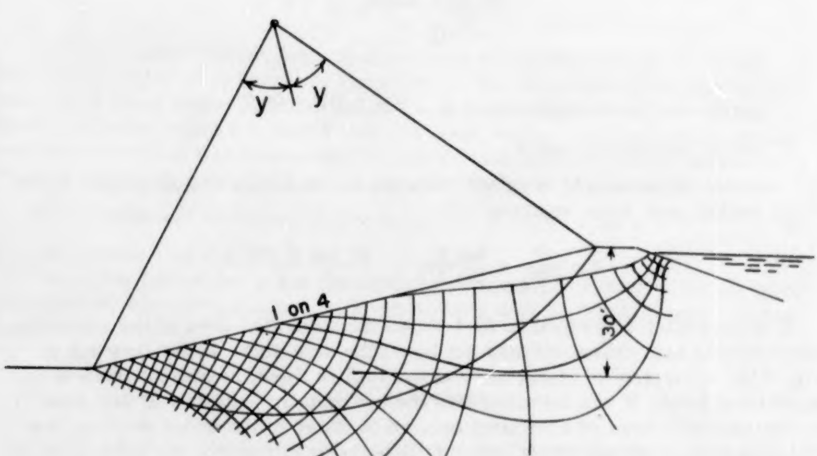
y
y
1 on 4
30°

Fig. 7

pressure due to a static column of water extending to the seepage line, thereby eliminating laborious graphical integration. The value of $\frac{wH \tan \phi}{c' \sin i \cos i}$ is then used to find the corresponding y in Fig. 5. When a value of y equal to the trial value is found from this computation, the wetted area may be planimetered, and a trial value of F may be found by means of equation (27). The exact method should be used for a final value, however, when the points of emergence of the critical arc have finally been established.

That the effect of seepage may be great is illustrated by problems (a) and (b) below. In these two problems idential slopes and identical materials have been assumed. Development of seepage through the slope results in a decrease of the safety factor from 2.18 to .701, which means failure.

Typical Problems

(a) A flat, uniform slope in homogeneous material. The rupture arc is assumed to pass through the edge of the crown and the toe. Slope is 1 on 4; $i = 14^0$; $H = 30$ feet. Laboratory tests have established that w is 100 lb/cu ft; $c = 300$ lb/sq ft; and $\phi = 15^0$.

$$K = \frac{c}{wH} = \frac{300}{100 \cdot 30} = 0.10$$

$\tan \phi = .26795$; $\tan i = .25$; $\sin i = .24192$; $\cos i = .9703$

$$\frac{\tan \phi}{K \sin i \cos i} = \frac{.26795}{.10 \cdot .24192 \cdot .9703} = 11.4$$

$y = 48^0$ (from Fig. 5)
$f_1 = .110$ (from Fig. 2)
$f_2 = 1.185$ (from Fig. 4)
$F = .10/.110 + 1.185 \cdot .26795/.25 = .91 + 1.27 = 2.18$

(b) (Fig. 7) A levee or earth dam with seepage established. Landside slope is 1 on 4; $i = 14^0$; $H = 30$ feet. Laboratory tests have established that $w = 100$ lb/cu ft; $c = 300$ lb/sq ft dry and 100 lb/sq ft saturated; and $\phi = 15^0$. A seepage flow net has been constructed as shown in Fig. 7. The rupture arc is assumed to pass through the near edge of the crown and the toe of the slope.

$\tan \phi = .26795$; $\tan i = .25$; $\sin i = .24192$; $\cos i = .9703$

(1) Assume $y = 40^0$. The arc is drawn and the dry and wet lengths of arc are scaled off; the dry and wet areas are planimetered. The dry length of arc is 10 feet and the wet length is 125 feet.

$y' = 40^0 \cdot 125/135 = 37^0$
$c' = (300 \cdot 3 + 100 \cdot 37)/40 = 115$ lb/sq ft

$$\frac{wH \tan \phi}{c' \sin i \cos i} = \frac{100 \cdot 30 \cdot .26795}{115 \cdot .24192 \cdot .9703} = 29.7$$

Then from Fig. 5, y should be 39^0.

(2) Assume $y = 39^0$. The new arc is drawn and the new c' is determined as above. The dry arc is again found to be 10 feet and the wet arc is 124 feet.

$y' = 39^0 \cdot 124/134 = 36^0$
$c' = (300 \cdot 3 + 100 \cdot 36)/39 = 115.4$ lb/sq ft

$$\frac{wH \tan \phi}{cL \sin i \cos i} = 29.6$$

From Fig. 5, y is still 39°.

The dry portion of the segment is found to be 149 sq ft; the wet portion is 1708 sq ft. Then by the approximate method, U = 106,580 lb.

$$F = \frac{115.4}{100 \cdot 30 \cdot .097} + \frac{.26797}{.25} 1.114 - \frac{6 \cdot 106{,}580 \cdot .26797 \cdot .24192}{100 \cdot 900 \cdot .62932}$$
$$= .397 + 1.194 - .732 = .859$$

On the other hand, when the solution is by the more exact method described above, the following table gives the value of U:

Arc, from toe feet	Mean head feet	Pressure lb/sq ft	Uplift lb
0-10	3.5	218	2,180
10-20	10.0	624	6,240
20-30	16.5	1,030	10,300
30-40	22.0	1,373	13,730
40-50	25.5	1,591	15,910
50-60	26.5	1,654	16,540
60-70	25.5	1,591	15,910
70-80	23.5	1,466	14,660
80-90	20.5	1,279	12,790
90-100	16.5	1,030	10,300
100-110	11.0	686	6,860
110-120	6.0	374	3,740
120-124	1.5	94	376
		U =	129,536

Then:

$$F = .397 + 1.194 - \frac{6 \cdot 129{,}536 \cdot .26797 \cdot .24192}{100 \cdot 900 \cdot .62932} = 1.591 - .890 = .701$$

ACKNOWLEDGMENT

The writer wishes to express his indebtedness to D. W. Taylor, whose paper: "Stability of Earth Slopes," (Journal of the Boston Society of Civil Engineers, July 1937), furnished the inspiration and some of the basis of this development. The original studies from this method was developed were made in collaboration with Mr. Robert M. German at the Soil Mechanics Laboratory of the Waterways Experiment Station, at Vicksburg, Mississippi.

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CONSOLIDATED CBR CRITERIA

R. G. Ahlvin,[1] M. ASCE
(Proc. Paper 1825)

SUMMARY

Analysis of all available service-behavior data from test sections and prototype airfields indicates that single-wheel CBR design criteria can be expressed in two basic parameters, $\frac{t}{\sqrt{A}}$ and $\frac{CBR}{p}$. These parameters can be reduced to a single plotted curve that separates service-behavior data with regard to failures and non-failures. The curve also represents the complete pattern of basic-strength requirements for flexible airfield pavements for single-wheel loadings. Previous work has demonstrated methods of reducing multiple-wheel loadings to their equivalent single-wheel loading.

The analysis has also shown that the upper elements of pavements should include an extra thickness for durability and for reducing differential settlement. No mathematical expression has yet been developed for this extra-thickness relation.

INTRODUCTION

Background

Since the very beginning of the development of the CBR flexible pavement design method by the Corps of Engineers, efforts have been made to express the CBR design curves as a mathematical equation. Such an equation is desirable for two purposes. First, it permits ready interpolation and extrapolation for the production of CBR design curves for new conditions which are perennially necessitated by the continuing development of aircraft. Second, it permits easy comparison of all the service-behavior data with the empirically

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developed CBR design curves for ready evaluation of the proper positioning of the empirical design curves.

Previous and Current Work

The late T. A. Middlebrooks of the Office, Chief of Engineers, first showed, in unpublished work, a strong relationship between the pattern of CBR design relations and that of theoretical shear stresses determined for an idealized loading on an elastic subgrade. Later, Fergus[1] presented an equation relating design thickness to wheel load for loads having a given contact or tire pressure. He showed that the constant relating these parameters was related to the CBR and to theoretical stress relations. In 1953, Kerkhoven and Dormon[2] gave expressions derived to fit the patterns of CBR design curves then used by the Corps of Engineers for design. More recently Turnbull and Ahlvin[3] showed that the equation presented by Fergus could be extended to consider the contact pressure variable and to incorporate the CBR directly.

This paper presents developments in the consolidation of the CBR system to date and shows that a comprehensive consolidated CBR system is emerging from the past and current work.

CBR-Load-Thickness Relationships

Extension of Mathematical Relationships

The mathematical expression of CBR relations presented in the paper by Turnbull and Ahlvin[3] gives the following equation:

$$t = \sqrt{\frac{P}{8.1\ \mathrm{CBR}} - \frac{A}{\pi}}$$

where

t = design thickness in inches

P = single-wheel load in pounds

A = tire contact area in square inches.

This expression represents the design criteria in current use for CBR values lower than about 15. For higher CBR values, the expression gives design thicknesses less than those specified for current design requirements. Thus, the CBR formula has only been applied in the lower CBR range.

The four variables in the CBR equation can, by rearrangement in the expression, be combined into only two. To accomplish this it is considered that the load P is equal to the product of the tire (or contact) pressure p and the tire contact area A. The anomalies or inconsistencies in this assumption have negligible effect on design thicknesses. The modified equation is as follows:

$$\frac{t}{\sqrt{A}} = \sqrt{\frac{1}{8.1 \times \frac{\mathrm{CBR}}{p}} - \frac{1}{\pi}}$$

where

p = tire pressure (or average contact pressure) in $\mathrm{lbs/in^2}$.

The equation in this form represents a single curve on a plot of the two composite parameters; see Fig. 1.

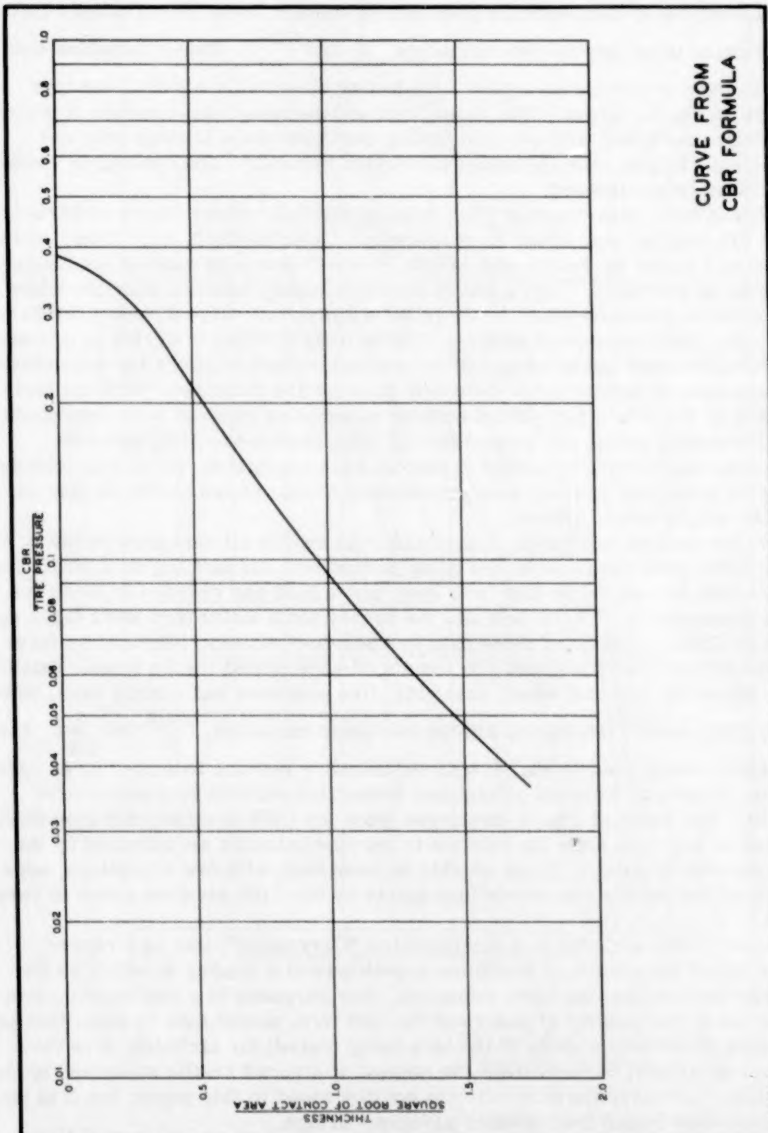

CURVE FROM
CBR FORMULA
CBR
TIRE PRESSURE
THICKNESS
SQUARE ROOT OF CONTACT AREA
0.01
0.02
0.03
0.04
0.05
0.06
0.08
0.1
0.2
0.3
0.4
0.5
0.6
0.8
1.0
0
0.5
1.0
1.5
2.0

FIGURE 1

Comparison of Mathematical Relationship with Service-Behavior Data

Since the effective parameters in a single-wheel loading test are design thickness, CBR, load, and tire pressure or contact area, it is a simple matter to combine them into the two variables, $\frac{t}{\sqrt{A}}$ and $\frac{CBR}{p}$. These variables then can be used to determine a point for plotting directly on a plot of the type shown in Fig. 1. Much of the recent test and service-behavior data, however, has been concerned with aircraft having multiple-wheel landing gear assemblies. In this case the added parameter of number and spacing of wheels must also be considered.

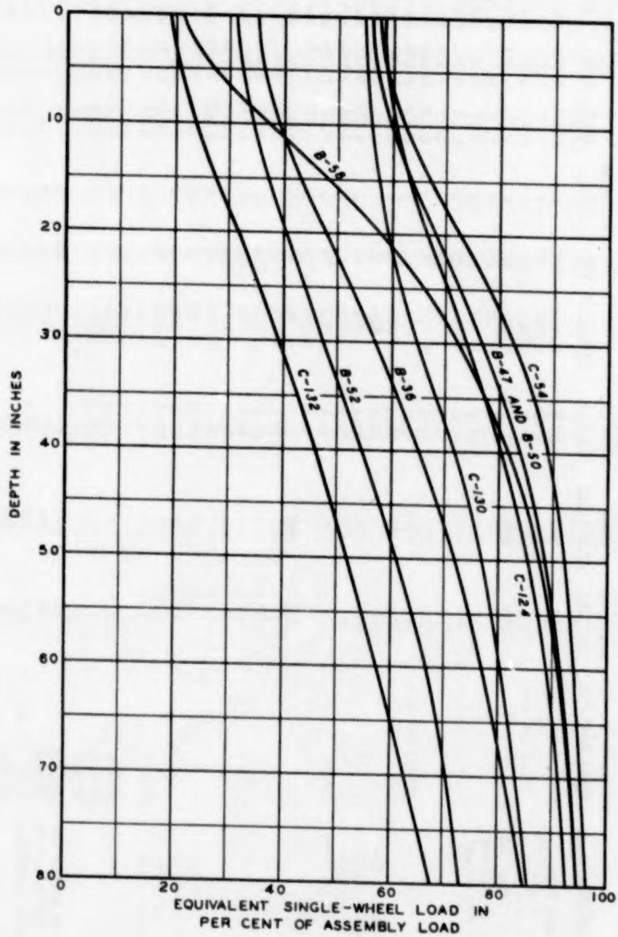
Means have been developed for relating multiple-wheel design criteria to CBR criteria for equivalent single wheels. These methods were reported in detail in a paper by Foster and Ahlvin,(4) which was presented at the Chicago meeting in February. Fig. 2 shows the relationship between multiple-wheel and equivalent single-wheel loadings for a number of current, heavy military aircraft. Relationships of this type can be used directly to arrive at equivalent single-wheel loads for any of the multiple-wheel loadings for which test-section data or field service-behavior records are available. With the test-section or field data for multiple-wheel assemblies reduced to an equivalent single-wheel loading, the parameters of load, thickness, CBR, and tire pressure can be used to arrive at the two basic variables, which will then determine points for plotting on a plot similar to that shown in Fig. 1, just as for the single-wheel criteria.

By the method described, it is possible to reduce all data used in the past to validate CBR relations to two basic parameters for plotting on a single plot. Therefore, the available data have been assembled and reduced to these two basic parameters. These data and the source from which they were taken appear in Table 1. Some of these data are service-behavior information from actual airfields, and the rest are results of accelerated traffic tests. Table 1 also shows the aircraft wheel assembly, tire pressure and contact area, wheel load, CBR, design thickness, and the two basic variables, $\frac{CBR}{p}$ and $\frac{t}{\sqrt{A}}$. These variables are plotted in Fig. 3, with satisfactory service indicated by an open circle, failure by a closed circle, and borderline service by a half-closed circle. The curve of Fig. 1, developed from the CBR equation, has also been plotted in Fig. 3 to show its relation to service behavior as indicated by the various plotted points. It can quickly be seen that, with few exceptions, separation of the failure and non-failure points by the CBR equation curve is very good.

Table 1 also includes a column entitled "Coverages"; this is a representation of the amount of traffic or repetitions of a loading to which an airfield or test section has been subjected. For purposes of a test section, one coverage is the number of passes of the test tire, placed side by side, that are required to cover the width of the lane being tested; for airfields, it is the equivalent of this, derived from the amount of aircraft traffic sustained by the airfield. The coverage parameter is not discussed in this paper, but it is important to any broad treatment of pavement design.

Comparison of Mathematical Relationship with Existing Criteria

It has been shown that in a plot of service-behavior data from test sections and in-service airfields reduced to the parameters of $\frac{t}{\sqrt{A}}$ and $\frac{CBR}{p}$, the failures



RELATIONS BETWEEN MULTIPLE-WHEEL AND EQUIVALENT SINGLE-WHEEL LOADS

R022657-A

FIGURE 2

Table 1

Service Behavior Data

Source of Data	Wheel Assembly	Contact Pressure or Inflation Pressure psi (p)	Contact Area sq in. (A)	Equivalent Single Wheel Load kip (P)	Coverages	CBR	Thickness in.	CBR/p	$t/\sqrt{A}$	Behavior
Design of Upper Base Courses for High-pressure tires; Report No. 1, Base Course Requirements as Related to Contact Pressures, TM 3-373	Twin, 37 in. c-c	100	267	33.8	2,000	102	1	0.80	0.061	Satisfactory
				33.8	2,000	98	1	0.77	0.061	Satisfactory
				33.8	2,000	55	1	0.43	0.061	Failure
		170	270	51.2	40	73	1	0.38	0.061	Satisfactory
				51.2	40	65	1	0.34	0.061	Satisfactory
				51.2	40	51	1	0.27	0.061	Failure
				51.2	40	35	1	0.18	0.061	Failure
				51.4	40	55	3	0.29	0.184	Satisfactory
				51.4	40	41	3	0.21	0.184	Failure
				51.4	40	33	3	0.17	0.184	Failure
				51.2	800	89	1	0.46	0.061	Satisfactory
				51.2	800	66	1	0.34	0.061	Failure
				51.2	800	40	1	0.21	0.061	Failure
				51.4	800	50	3	0.26	0.184	Satisfactory
				51.4	2,000	79	3	0.41	0.184	Satisfactory
				51.4	2,000	53	3	0.28	0.184	Failure
		224	267	67.6	40	58	1	0.23	0.061	Failure
				67.6	40	49	1	0.19	0.061	Failure
				67.6	40	44	1	0.17	0.061	Failure
				67.6	40	41	1	0.16	0.061	Failure
				67.8	40	112	3	0.44	0.184	Satisfactory
				67.8	40	66	3	0.26	0.184	Failure
				67.8	40	38	3	0.15	0.184	Failure
				67.6	800	83	1	0.33	0.061	Satisfactory
				67.6	2,000	88	1	0.35	0.061	Satisfactory
				67.6	2,000	40	1	0.16	0.061	Failure
				67.8	2,000	88	3	0.35	0.184	Failure
				67.8	2,000	85	3	0.34	0.184	Satisfactory
				67.8	2,000	68	3	0.27	0.184	Satisfactory
Investigation of the Design and Control of Asphalt Paving Mixtures, TM 3-254	Single	54	278	15	3,500	28	9	0.52	0.54	Satisfactory
	Single	99	374	37	1,500	31	13	0.31	0.673	Satisfactory
	Twin, 37 in. c-c	118	329	38.7	1,500	35	16	0.30	0.88	Satisfactory
	Single	54	278	15	3,500	40	9	0.74	0.54	Satisfactory
	Single	99	374	37	1,500	34	13	0.34	0.673	Satisfactory
	Twin, 37 in. c-c	118	329	38.7	1,500	43	16	0.36	0.88	Satisfactory
	Single	54	278	15	3,500	33	9	0.61	0.54	Satisfactory
	Single	99	374	37	1,500	37	13	0.37	0.673	Satisfactory
	Twin, 37 in. c-c	118	329	38.7	1,500	31	16	0.26	0.88	Satisfactory

	Single	54	278	15	3,500	29	9	0.54	0.54	Satisfactory
	Twin, 37 in. c-c	118	329	38.7	666	41	16	0.35	0.88	Satisfactory
	Twin, 37 in. c-c	118	329	38.7	666	30	16	0.25	0.88	Satisfactory
	Single	99	374	37	100	33	11	0.33	0.569	Satisfactory
	Single	99	374	37	100	14	11	0.14	0.569	Failure
	Twin, 37 in. c-c	110	329	36.3	100	27	11	0.25	0.61	Satisfactory
	Twin, 37 in. c-c	110	329	36.3	100	13	2	0.12	0.11	Failure
	Single	99	374	37	400	18	13	0.18	0.673	Satisfactory
	Single	99	374	37	400	10	13	0.10	0.673	Failure
	Single	99	374	37	400	13	13	0.13	0.673	Failure
	Single	99	374	37	400	5	16	0.05	0.828	Failure
	Single	99	374	37	350	4	16	0.04	0.828	Failure
	Single	99	374	37	260	2	16	0.02	0.828	Failure
Service Behavior Test Section, Barksdale Field, Louisiana	Single	63.6	314	20	250	5	10.5	0.079	0.59	Failure
		63.5	315	20	500	5	13	0.079	0.73	Failure
		63.5	315	20	1,000	5	15.5	0.079	0.87	Failure
		56.6	353	20	3,000	5	17.5	0.088	0.93	Failure
		56.6	353	20	5,000	5	18	0.088	0.95	Failure
		70.7	707	50	200	5.5	17.5	0.078	0.66	Failure
		70.7	707	50	500	5.5	20.5	0.078	0.77	Failure
		70.7	707	50	1,000	5.5	24	0.078	0.90	Failure
		80.4	622	50	3,000	5.5	26	0.068	0.98	Failure
		80.4	622	50	5,000	5.5	26.5	0.068	1.00	Failure
Investigation of Effects of Traffic with High-pressure Tires on Asphalt Pavements, TM 3-312	Single	200	150	30	216	14	12	0.07	0.98	Failure
		200	150	30	178	7	12	0.035	0.98	Failure
		200	150	30	178	7	12	0.035	0.98	Failure
		200	150	30	203	6	12	0.03	0.98	Failure
	Twin tandem, 31x63 in. c-c	200	150	36	32	36	1.5	0.150	0.122	Failure
		200	150	36	542	65	2	0.271	0.163	Failure
		200	150	43.2	2,001	35	11	0.122	0.897	Satisfactory
Design of Flexible Airfield Pavements for Multiple-wheel Landing Gear Assemblies; Report No. 1, Test Section with Lean Clay Subgrade, TM 3-349	Twin, 37 in. c-c	91	330	42.9	2,000	35	10	0.27	0.55	Satisfactory
		91	330	45.6	2,000	25	15	0.20	0.82	Satisfactory
		91	330	48.6	2,000	20	20	0.14	1.10	Satisfactory
	Twin tandem, 31x63 in. c-c	140	267	63.5	2,000	34	14	0.14	0.86	Satisfactory
		140	267	71.3	2,000	29	20	0.11	1.22	Satisfactory
		140	267	79.5	2,000	22	26	0.074	1.59	Satisfactory
		187	267	84.6	610	25	14	0.079	0.86	Failure
		187	267	95.0	2,000	27	20	0.076	1.22	Borderline
		187	267	106.0	2,000	20	26	0.050	1.59	Satisfactory
	Twin, 37 in. c-c	187	267	59.5	328	11	10	0.049	0.61	Failure
		187	267	63.5	2,000	23	15	0.097	0.92	Borderline
		187	267	68.0	2,000	30	20	0.117	1.22	Satisfactory

(Continued)

Table 1 (Continued)

Source of Data	Wheel Assembly	Contact Pressure or Inflation Pressure psi (p)	Contact Area sq in. (A)	Equivalent Single Wheel Load kip (P)	Coverages	CBR	Thickness in.	CBR/p	$t/\sqrt{A}$	Behavior
Report on Stockton Runway Test Section, Sept 1942	Single	65	385	25	200	4	12	0.061	0.61	Failure
		65	385	25	300	4	14.5	0.061	0.74	Failure
		65	385	25	500	4	18	0.061	0.92	Failure
		65	385	25	1,000	4	22	0.061	1.12	Failure
		65	385	25	2,000	4	24.5	0.061	1.25	Failure
		65	385	25	3,000	4	25.0	0.061	1.28	Failure
		72	556	40	200	4	20	0.055	0.85	Failure
		72	556	40	500	4	26	0.055	1.10	Failure
		72	556	40	1,000	4	31	0.055	1.32	Failure
		72	556	40	2,000	4	36	0.055	1.53	Borderline
		72	556	40	3,000	4	38	0.055	1.61	Borderline
Accelerated Traffic Test at Stockton Airfield, Stockton, California, Test No. 2	Single	133	1,501	200	150	6	39	0.045	1.01	Failure
		133	1,501	200	1,700	9	44	0.068	1.14	Failure
		133	1,501	200	2,000	10	46.5	0.075	1.20	Borderline
		133	1,501	200	2,000	9	51	0.068	1.32	Satisfactory
		133	1,501	200	2,000	7	60.5	0.053	1.56	Satisfactory
		133	1,501	200	10	14	18	0.105	0.46	Failure
		133	1,501	200	60	16	20.5	0.120	0.53	Failure
		133	1,501	200	360	13	24.5	0.098	0.63	Failure
		133	1,501	200	1,500	13	30	0.098	0.77	Failure
		133	1,501	200	800	15	34	0.113	0.88	Failure
		133	1,501	200	2,500	75	9.5	0.564	0.25	Satisfactory
		133	1,501	200	2,000	75	10.5	0.564	0.27	Satisfactory
		133	1,501	200	2,000	75	13.5	0.564	0.35	Satisfactory
		133	1,501	200	2,000	75	9.5	0.564	0.25	Satisfactory
		133	1,501	200	2,500	75	13.5	0.564	0.35	Satisfactory
		133	1,501	200	2,000	75	15	0.564	0.39	Satisfactory
		133	1,501	200	2,500	75	21	0.564	0.54	Satisfactory
		133	1,501	200	2,000	75	16	0.564	0.41	Satisfactory
		133	1,501	200	2,000	11	66	0.083	1.70	Satisfactory
		133	1,501	200	2,000	75	22	0.564	0.57	Satisfactory
		133	1,501	200	1,150	10	52	0.075	1.34	Satisfactory
		133	1,501	200	1,300	8	30	0.060	0.77	Failure
		133	1,501	200	850	75	8	0.564	0.21	Failure
		133	1,501	200	2,000	85	9	0.639	0.23	Satisfactory
		133	1,501	200	2,000	75	23.5	0.564	0.61	Satisfactory
		133	1,501	200	2,000	85	23.5	0.639	0.61	Satisfactory

Flexible Pavement Behavior Studies, Interim Report No. 2										
Berry	Single	70	214	15	1,741	23	12	0.329	0.82	Satisfactory
		70	214	15	1,741	43	12	0.614	0.82	Satisfactory
		70	214	15	1,741	27	12	0.386	0.82	Satisfactory
		70	214	15	1,741	16	12	0.229	0.82	Satisfactory
Dodge	Single	66	226	15	4,344	17	4	0.256	0.226	Failure
Douglas	Single	67	260	17.5	284	27	20	0.435	1.24	Satisfactory
		67	260	17.5	3,380	24	19	0.387	1.18	Satisfactory
		67	260	17.5	3,380	30	19	0.484	1.18	Satisfactory
		67	260	17.5	3,380	24	15	0.387	0.928	Satisfactory
		67	260	17.5	3,380	24	14	0.387	0.866	Satisfactory
		67	260	17.5	3,380	38	14	0.612	0.866	Satisfactory
Jackson	Single	60	250	15	185	37	2	0.616	0.126	Satisfactory
		60	250	15	185	39	2	0.65	0.126	Satisfactory
Kirtland	Single	84	176	15	707	32	10.5	0.377	0.785	Satisfactory
	Single	84	360	30	1,225	59	11	0.707	0.579	Satisfactory
		84	360	30	12,250	64	10	0.767	0.526	Satisfactory
		84	360	30	12,250	47	11	0.563	0.579	Satisfactory
		84	360	30	12,250	36	11.5	0.432	0.605	Satisfactory
Pueblo	Single	84	360	30	1,470	17	11	0.204	0.579	Satisfactory
		84	360	30	1,470	16	11	0.192	0.579	Satisfactory
		84	360	30	1,060	25	12	0.300	0.632	Satisfactory
Santa Fe	Single	42	360	15	696	28	10	0.672	0.526	Satisfactory
Yuma	Single	84	360	30	22,500	44	13	0.527	0.684	Satisfactory
		84	360	30	22,500	50	12	0.599	0.632	Satisfactory
		84	360	30	22,500	38	14	0.455	0.737	Satisfactory
Lawson	Single	60	250	15	3,475	22	9.5	0.367	0.601	Satisfactory
		60	250	15	3,475	25	10	0.417	0.632	Satisfactory
		60	250	15	3,760	8	10	0.133	0.632	Failure
		60	250	15	3,760	9	10	0.150	0.632	Failure
Condition Survey, Report No. 5, Eglin Air Force Base, Valparaiso, Fla., MP 4-3	Single	67	260	15	150	59	5	0.88	0.309	Satisfactory
		67	260	15	150	18	5	0.269	0.309	Failure
		67	260	15	150	49	5	0.73	0.309	Satisfactory
		67	260	15	910	63	12	0.94	0.745	Satisfactory
		67	260	15	910	20	12	0.299	0.745	Satisfactory

(Continued)

Table 1 (Continued)

Source of Data	Wheel Assembly	Contact Pressure or Inflation Pressure psi (p)	Contact Area sq in. (A)	Equivalent Single Wheel Load kip (P)	Coverages	CBR	Thickness in.	CBR/p	$t/\sqrt{A}$	Behavior
Condition Survey; Report No. 7, Kirtland Air Force Base, Albuquerque, N. Mex., Surveys of 1945-1952, MP 4-3	Twin, 37 in. c-c	140	267	42.2	Cap.	48	2	0.303	0.122	Failure
		140	267	42.2	Cap.	103	2	0.652	0.122	Satisfactory
Condition Survey; Report No. 4, Ardmore Air Force Base, Ardmore, Okla., MP 4-3	Single	61	360	22	7,500	57	3	0.934	0.158	Borderline
Condition Survey; Report No. 3, Lawson Air Force Base, Ft. Benning, Ga., MP 4-3	Single	60	250	15	4,100	59	10	0.984	0.633	Satisfactory
		60	250	15	4,100	20	2	0.333	0.126	Borderline
		60	250	15	4,100	16	2	0.267	0.126	Failure
		60	250	15	4,100	21	2	0.350	0.126	Borderline
		60	250	15	2,700	18	2	0.300	0.126	Borderline
		60	250	15	2,700	23	2	0.384	0.126	Borderline
		60	250	15	4,100	39	11	0.65	0.695	Satisfactory
Unpublished data:										
Blythe	Single	69	360	25	3,380	60	8	0.869	0.421	Satisfactory
		69	360	25	3,380	65	8	0.942	0.421	Satisfactory
		69	360	25	3,380	66	8	0.956	0.421	Satisfactory
Gainesville	Single	69	360	25	2,310	6	32	0.087	1.63	Satisfactory
		69	360	25	578	13	26	0.189	1.37	Satisfactory
Field moisture: Keesler	Single	58	260	15	1,680	36	13	0.620	0.807	Satisfactory
		58	260	15	840	44	9	0.759	0.559	Satisfactory
Unpublished data: Las Vegas	Single	84	360	30	8,510	24	13	0.288	0.685	Satisfactory
		84	360	30	5,560	23	16	0.275	0.842	Satisfactory
		84	360	30	5,560	31	16	0.371	0.842	Satisfactory
		84	360	30	5,560	14	16	0.167	0.842	Satisfactory
		84	360	30	7,990	47	11	0.567	0.579	Satisfactory
Design of Upper Base Courses for High-pressure Tires, Report No. 2 (Unpublished)	Twin, 37 in. c-c	170	267	51.6	2,000	111	1.5	0.575	0.092	Satisfactory
		170	267	51.7	850	34	2.5	0.174	0.153	Failure
		170	267	51.7	850	105	2.5	0.538	0.153	Satisfactory
		170	267	51.7	700	29	2.5	0.149	0.153	Failure

	170	267	51.7	850	90	2.5	0.461	0.153	Satisfactory
	170	267	51.7	700	41	2.5	0.210	0.153	Borderline
	170	267	51.7	700	47	2.5	0.241	0.153	Failure
	170	267	51.7	50	22	2.5	0.113	0.153	Failure
	170	267	51.9	2,000	165+	3.5	0.849	0.214	Satisfactory
	170	267	52.2	500	21	4.5	0.107	0.275	Failure
	170	267	52.2	700	140+	4.5	0.717	0.275	Satisfactory
	170	267	52.2	60	14	4.5	0.072	0.275	Failure
	170	267	52.2	522	67	4.5	0.343	0.275	Satisfactory
	170	267	52.2	22	14	4.5	0.072	0.275	Failure
	170	267	52.2	40	20	4.5	0.102	0.275	Failure
	170	267	52.8	2,000	157+	6.5	0.794	0.398	Satisfactory
Single	300	200	60	800	158+	6.5	0.527	0.460	Satisfactory
Twin, 37 in. c-c	170	267	53.3	2,000	103	7.5	0.516	0.459	Satisfactory
Single	300	200	60	800	60	7.5	0.200	0.530	Borderline
Twin, 37 in. c-c	170	267	53.3	800	29	7.5	0.145	0.459	Failure
	170	267	53.3	2,000	59	7.5	0.295	0.459	Satisfactory
Single	300	200	60	800	23	7.5	0.077	0.530	Failure
Twin, 37 in. c-c	170	267	53.3	60	6	7.5	0.030	0.459	Failure
	170	267	53.3	220	23	7.5	0.077	0.459	Failure
	170	267	53.3	2,000	28	7.5	0.140	0.450	Borderline
Single	300	200	60	700	10	7.5	0.033	0.530	Failure

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and non-failures are well separated by the CBR equation curve. It becomes of interest then to examine the pattern of present criteria in consolidated form. Accordingly, a number of families of CBR curves, which plot thickness versus CBR for a range of load curves, have been combined by reducing their variables to the two basic parameters of $\frac{t}{\sqrt{A}}$ and $\frac{CBR}{p}$. These have been placed on a plot similar to those shown in Figs. 1 and 3. The resulting plot is shown in Fig. 4. The curve from the mathematical relation has been added for comparison. Good agreement is shown for all points representing CBR values below about 12 (indicated by circles). For CBR values above 12 (indicated by triangles), existing criteria show greater thickness requirements than does the CBR formula curve.

Discussion of Minimum Thickness

The greater thickness requirements of existing design curves in the higher CBR range reflect a need for other than minimum supporting strength in the upper elements of a pavement. This extra thickness has been provided for durability and to reduce differential settlement. The latter reason for extra thickness was shown in a paper by Foster(5) in 1949. These requirements are probably dependent on magnitude of the load but are not related to loading in the same way as are CBR requirements.

In existing criteria the special, extra-thickness requirements in the top elements of a pavement structure and the deeper, basic-strength requirements are combined and blended into smooth curves. This was an automatic blending growing from the empirical development of the design curves, and it is only as a result of the analysis presented here that separation of design criteria into two parts is possible. The pattern of the deeper, basic-strength requirements is represented by the mathematical expression of CBR relations. The extra-strength requirements in the top elements of a pavement can be specified in terms of minimum thicknesses of surfacing and base course of specified quality. No mathematical expression has yet been established for minimum base and pavement thicknesses, but the pattern of requirements specified by the Corps of Engineers is reflected in Table 2.

Summation

From the analysis presented, it is concluded that single-wheel CBR design criteria can be expressed in two basic parameters, $\frac{t}{\sqrt{A}}$ and $\frac{CBR}{p}$, and thereby reduced to a single plotted curve. It has also been shown that this curve separates service-behavior data, plotted in the same parameters, with regard to failures and non-failures. Thus, it may be concluded that the mathematical CBR expression, or the single curve derived from it, in the two basic variables, $\frac{t}{\sqrt{A}}$ and $\frac{CBR}{p}$, represents the complete pattern of basic-strength requirements for flexible airfield pavements for single-wheel loadings. Also, previously reported work(4) relating multiple-wheel loadings to equivalent single-wheel loadings has demonstrated means of reducing any loading to its equivalent single-wheel loading. This brings all loadings under the purview of the CBR equation.

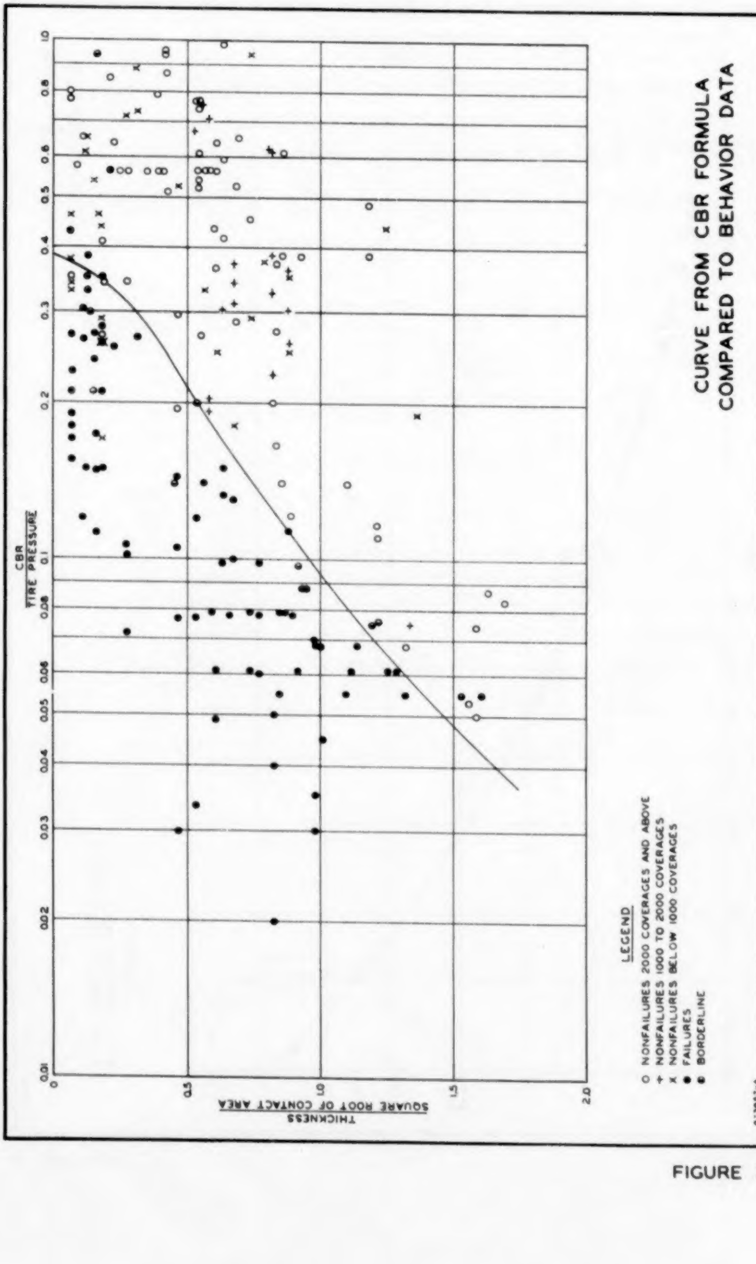
CURVE FROM CBR FORMULA
COMPARED TO BEHAVIOR DATA
CBR / TIRE PRESSURE
THICKNESS / SQUARE ROOT OF CONTACT AREA
0.01
0.02
0.03
0.04
0.05
0.06
0.08
0.1
0.2
0.3
0.4
0.5
0.6
0.8
1.0
0
0.5
1.0
1.5
2.0
LEGEND
NONFAILURES 2000 COVERAGES AND ABOVE
NONFAILURES 1000 TO 2000 COVERAGES
NONFAILURES BELOW 1000 COVERAGES
FAILURES
BORDERLINE

FIGURE 3

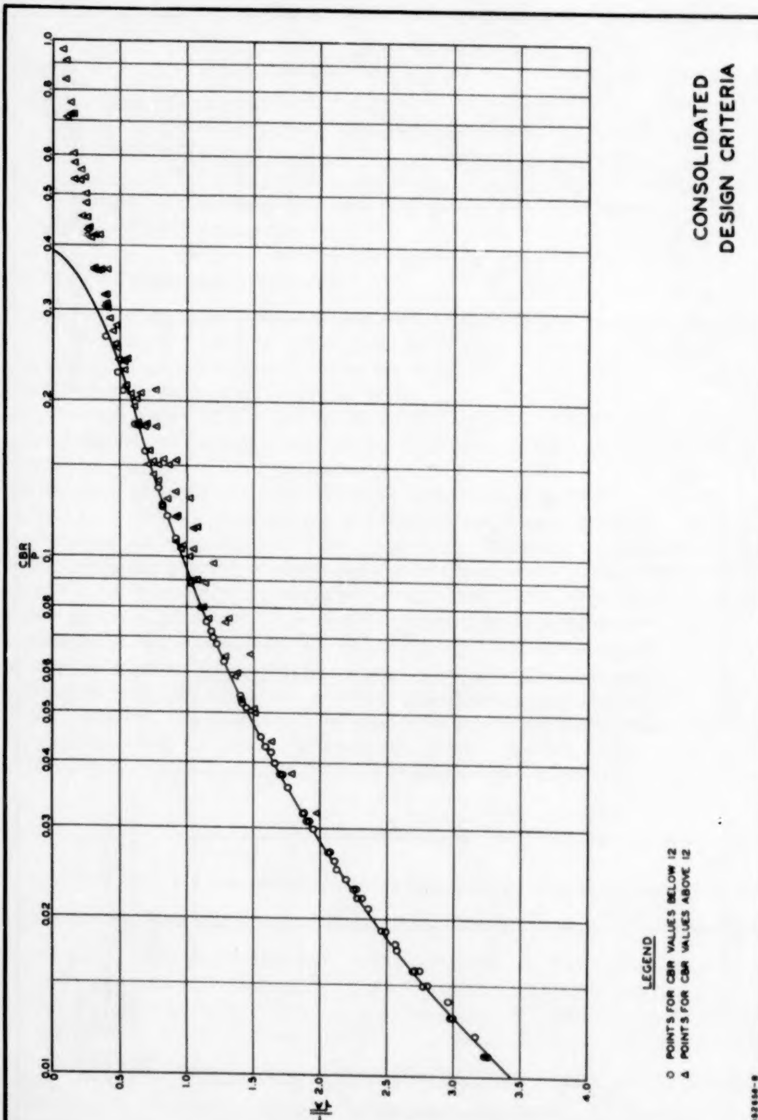
CONSOLIDATED
DESIGN CRITERIA
LEGEND
○ POINTS FOR CBR VALUES BELOW 12
△ POINTS FOR CBR VALUES ABOVE 12
CBR/p
0.01
0.02
0.03
0.04
0.05
0.06
0.08
0.1
0.2
0.3
0.4
0.5
0.6
0.8
1.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

FIGURE 4

Table 2

Minimum Allowable Thicknesses of Pavement and Base Course

Typical Aircraft	Wheel Assembly	Tire Pressure or Contact Area	Assembly Load kip	Thickness, in. 100-CBR Base Pavement	100-CBR Base Base	80-CBR Base Pavement	80-CBR Base Base
	Single	100 psi	10	2	6	2	6
			30	2	6	3	6
			50	2	6	3	6
	Single	100 sq in.	10	2	6	2	6
			20	3	6	3	6
			30	4	6	5	6
C-54	Twin, 28 in. c-c	225 sq in.	20	2	6	2	6
			60	2	6	3	6
			100	2	6	3	6
C-124	Twin, 44 in. c-c	630 sq in.	50	2	6	2	6
			100	2	6	3	6
			150	3	6	4	6
B-36	Twin-tandem, 31x63 in.	267 sq in.	100	2	6	3	6
			135	3	6	3	6
			170	3	6	3	6
B-47	Twin, 37 in. c-c	267 sq in.	75	3	6	3	6
			100	3	6	4	6
			125	4	8	5	7
B-52	Twin-twin, 37-62-37 in.	267 sq in.	200	3	7	4	6
			230	4	8	5	7
			265	4	9	5	8

The analysis has shown that strength requirements for the upper elements of a pavement include, in addition to basic supporting strength, extra thickness for durability and for reducing differential settlement. These extra-thickness requirements can be treated independently of those for basic supporting strength. No expression for the pattern of these extra-thickness relations has yet been developed.

ACKNOWLEDGMENTS

The work which forms the immediate basis for this report was carried on at the U. S. Army Engineer Waterways Experiment Station between 1954 and 1957 for the Office, Chief of Engineers, in connection with both Air Force and Army projects. Supervisory engineers reponsible for the conduct of the work include Messrs. W. J. Turnbull and C. R. Foster. Engineers directly associated with the work include Messrs. R. G. Ahlvin, D. N. Brown, and E. J. French (formerly with the Waterways Experiment Station).

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Paper 1826 October, 1958 SM 4

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

GLOSSARY OF TERMS AND DEFINITIONS IN SOIL MECHANICS

Report of the Committee on Glossary of Terms and Definitions in Soil Mechanics of the Soil Mechanics and Foundations Division
(Proc. Paper 1826)

PREFACE

Brief Resume of Committee Activities

On November 4, 1936, the Soil Mechanics and Foundations Division appointed a committee, ". . . . to prepare a glossary of terms and definitions on soil mechanics and to study, confer with other organizations, and report on classification of soils." The first committee, comprised of Harry Englander, Glennon Gilboy, W. S. Housel, R. V. Labarre, W. L. Shannon and W. P. Kimball chairman, recognized in the course of its study a persistent and growing demand among engineers for consistency in the use of symbols, as well as definitions. In recognition of this need, the Committee prepared a glossary of terms and definitions entitled "Soil Mechanics Nomenclature." This glossary was adopted by the Society on April 20, 1941, and published as "Manual of Engineering Practice, No. 22."

In submitting its report, this first Committee expressed the view that the publication of the glossary did not eliminate the need for continued effort. On the contrary, it recommended that it be reviewed periodically and revised as necessary to keep pace with future developments.

For the purpose of making a first revision, a new Committee was appointed in January 1947, consisting of W. H. Jervis, W. L. Shannon, S. R. Stearns and R. E. Fadum, chairman. In January 1948 the appointment of G. A. Leonards and that of E. A. Abdun-Nur were approved; in August 1950, T. W. Lambe was added; in January 1954, C. R. Foster replaced E. A. Abdun-Nur as the ASTM representative; and in January 1955, R. G. Ahlvin was added to the Committee.

Whereas the first glossary (Manual No. 22) placed major emphasis on symbology and definitions of those terms for which symbols are required, the new Committee was of the opinion that in revising the glossary major emphasis should be placed on definitions relating to all aspects of the subject. Accordingly, the Committee began its work by making a search of contemporary literature on soil mechanics. Some forty-two publications were

Note: Discussion open until March 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1826 is part of the copyrighted Journal of the Soil Mechanics Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 4, October, 1958.

examined[1] with a view towards obtaining an authoritative list of terms. Terms that appeared to be in need of definition, together with definitions as given by the various authors of these Publications, were compiled in a preliminary report dated August 1951.

The preliminary report was made available to Sub-committee G-3 on Nomenclature and Definitions of Committee D-18 of ASTM. This Subcommittee had been charged with a similar responsibility as the ASCE Committee and it was mutually agreed that efforts would be coordinated to the end that a glossary acceptable to both bodies would result. Since the ASCE Committee had already invested considerable effort in the preparation of the preliminary report, Subcommittee G-3 began its work with this report. The preliminary compilation was reviewed term by term. Suggested definitions were accepted or modified and many terms were added and some deleted. This effort by the ASTM Committee culminated in an interim-type report dated June 1954.

Concurrently, by similar but independent action, the ASCE Committee reached a consensus and prepared an interim report dated June 1953. It then submitted its interim report to a number of outside agencies and individuals for comment and review.

There were substantial differences in the ASCE and ASTM interim compilations. Accordingly, Committee Chairmen C. R. Foster, ASTM, and R. E. Fadum, ASCE, assisted by R. G. Ahlvin, combined the two works incorporating the suggestions of outside agencies and individuals that had previously been solicited. They then met several times in an effort to reconcile differences as necessary term by term. The resulting glossary was completed and reproduced in April 1956 and submitted to Subcommittee G-3 of Committee D-18, ASTM, and to the Committee on Glossary of Terms and Definitions in Soil Mechanics of the Soil Mechanics and Foundations Division of ASCE for approval.

Some of the Committee members expressed the view that the glossary needed still further revision. In response to this expression, two joint ASCE–ASTM Committee meetings were held at which the entire compilation was again reviewed and revised. This revision and some minor changes agreed to by correspondence have led to the glossary presented herewith.

Under date of May 13, 1958, the chairman was advised by the ASTM representative on the Committee that Committee D-18 had recommended ASTM acceptance of the glossary.

A review of the effort involved in this compilation clearly demonstrates the difficulty of reaching agreement on such a controversial subject as definitions. The Committee is, however, persuaded that such effort is justified and that only by a continuing study will it be possible by a process of evolution to narrow areas of disagreement. It therefore recommends that this glossary be published in the Journal of the Soil Mechanics and Foundations Division of the Society, whereby discussion will be invited. This action has been endorsed and formally approved by the Executive Committee of the Soil Mechanics and Foundations Division.

To focus attention on those terms the definitions of which are particularly controversial, it will be noted that some of the terms in the glossary are designated by an asterisk. It is hoped that these terms especially will receive full discussion.

1. See Bibliography appended to Glossary.

Explanatory Notes:

1. Dimensions, where applicable, are indicated in capital letters on the right-hand side under the item and immediately above the definition. The letters denote:

   F FORCE, such as pound, ton, gram, kilogram
   L LENGTH, such as inch, foot, centimeter
   T TIME, such as second, minute
   D DIMENSIONLESS

   Positive exponents designate multiples in the numerator. Negative exponents designate multiples in the denominator. Degrees of angle are indicated as "Degrees."

   Expressing the units either in the CGS or English system has been purposely omitted in order to leave the choice of the system and specific unit to the engineer and the particular application.

   For example:

   FL^{-2} - may be expressed in pounds per square inch, kilograms per square centimeter, tons per square foot, etc.

   LT^{-1} - may be expressed in feet per minute, centimeters per second, etc.

2. Words in quotes in the definitions refer to words that are defined elsewhere in the glossary. In the final printed work, these words are to be italicized.

3. Terms are included in alphabetical order and are numbered merely for ease of reference. It is not intended that the numbers will be included in the final printed work.

4. Where synonymous terms are cross referenced, the definition is usually included with the earlier term alphabetically. Where this is not the case, the latter term is the more significant.

5. No significance should be placed on the order in which symbols are presented where two or more are given.

1. AASHO COMPACTION

   See Compaction Test.

2. "A" HORIZON

   See Horizon.

*3. ABSORBED WATER

   Water held mechanically in a soil mass and having physical properties not substantially different from ordinary water at the same temperature and pressure.

4. ACTIVE EARTH PRESSURE

   See Earth Pressure.

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* Designates terms the definitions of which are particularly controversial.

5. ACTIVE STATE OF PLASTIC EQUILIBRIUM

See Plastic Equilibrium.

6. ADHESION

Unit: c_a	FL^{-2}
Total: C_a	F or FL^{-1}

"Shearing resistance" between "soil" and another material under zero externally applied pressure.

*7. ADSORBED WATER

Water in a "soil" mass, held by physico-chemical forces, having physical properties substantially different from "absorbed water" or chemically combined water, at the same temperature and pressure.

8. AEOLIAN DEPOSITS

Wind-deposited material such as dune sands and "loess" deposits.

9. AIR-SPACE RATIO

G_a	D

Ratio of (1) volume of water that can be drained from a saturated "soil" under the action of force of gravity to (2) total volume of "voids."

10. AIR-VOID RATIO

G_v	D

The ratio of (1) the volume of air space to (2) the total volume of "voids" in a "soil" mass.

11. ALLOWABLE BEARING VALUE (ALLOWABLE SOIL PRESSURE)

q_a, p_a	FL^{-2}

The maximum pressure that can be permitted on foundation "soil," giving consideration to all pertinent factors, with adequate safety against rupture of the "soil" mass or movement of the "foundation" of such magnitude that the structure is impaired.

12. ALLOWABLE PILE BEARING LOAD

Q_a, P_a	F

The maximum load that can be permitted on a "pile" with adequate safety against movement of such magnitude that the structure is endangered.

13. ALLUVIUM

"Soil" the constituents of which have been transported in suspension by flowing water and subsequently deposited by sedimentation.

14. ANGLE OF EXTERNAL FRICTION (ANGLE OF WALL FRICTION)

δ	Degrees

Angle between the abscissa and the tangent of the curve representing the relationship of "shearing resistance" to "normal stress" acting between "soil" and surface of another material.

*15. ANGLE OF INTERNAL FRICTION

ϕ Degrees

Angle between the abscissa and the tangent of the curve representing the relationship of "shearing resistance" to "normal stress" acting within a "soil."

16. ANGLE OF OBLIQUITY

α, B, θ, Ψ Degrees

The angle between the direction of the resultant "stress" or force acting on a given plane and the normal to that plane.

17. ANGLE OF REPOSE

α Degrees

Angle between the horizontal and the maximum slope that a soil assumes through natural processes. For dry granular "soils" the effect of the height of slope is negligible; for "cohesive soils" the effect of height of slope is so great that the "angle of repose" is meaningless.

18. ANGLE OF WALL FRICTION

See Angle of External Friction.

19. ANISOTROPIC MASS

A mass having different properties in different directions at any given point.

20. APPARENT COHESION

See Cohesion.

21. AQUIFER

A water-bearing formation that provides a "ground water" reservoir.

22. ARCHING

The transfer of "stress" from a yielding part of a "soil" mass to adjoining less-yielding or restrained parts of the mass.

23. AREA OF INFLUENCE OF A WELL

a L^2

Area surrounding a well within which the "piezometric surface" has been lowered when pumping has produced the maximum steady rate of flow.

24. AREA RATIO OF A SAMPLING SPOON, SAMPLER, OR SAMPLING TUBE

A_r D

$A_r = \frac{D_e^2 - D_i^2}{D_i^2} \times 100$ where D_e represents the

maximum external diameter of the sampling spoon and D_i represents the minimum internal diameter of the sampling spoon at the cutting edge. The area ratio is an indication of the volume of "soil" displaced by the sampling spoon (tube).

25. "B" HORIZON

See Horizon.

26. BASE COURSE (BASE)

A layer of specified or selected material of planned thickness constructed on the "subgrade" or "subbase" for the purpose of serving one or more functions such as distributing load, providing drainage, minimizing frost action, etc.

*27. BASE EXCHANGE

The physico-chemical process whereby one species of ions adsorbed on "soil" particles is replaced by another specie.

28. BEARING CAPACITY

See Ultimate Bearing Capacity.

29. BEARING CAPACITY (OF A PILE)

Q_p, P_p F

The load per "pile" required to produce a condition of failure.

30. BEDROCK (LEDGE)

Rock of relatively great thickness and extent in its native location.

*31. BENTONITIC CLAY

A "clay" with a high content of the mineral montmorillonite, usually characterized by high swelling on wetting.

32. BERM

A shelf that breaks the continuity of a slope.

33. BINDER (SOIL BINDER)

Portion of soil passing No. 40 U. S. standard sieve.

34. BOULDER

A rock fragment, usually rounded by weathering or abrasion, with an average dimension of 12 in. or more.

35. BOULDER CLAY

A geological term used to designate glacial drift that has not been subjected to the sorting action of water and therefore contains particles from boulders to "clay sizes."

36. BULB OF PRESSURE

See Pressure Bulb.

37. BULKING

The increase in volume of a material due to manipulation. "Rock" bulks upon being excavated; damp "sand" bulks if loosely deposited, as by dumping, because the "apparent cohesion" prevents movement of the "soil" particles to form a reduced volume.

38. BUOYANT UNIT WEIGHT (SUBMERGED UNIT WEIGHT)

See Unit Weight.

39. "C" HORIZON

See Horizon.

40. CALIFORNIA BEARING RATIO

CBR D

The ratio of (1) the force per unit area required to penetrate a "soil" mass with a 3-sq-in. circular piston (approximately 2-in. diameter) at the rate of 0.05 in. per min to (2) that required for corresponding penetration of a standard material. The ratio is usually determined at 0.1-in. penetration, although other penetrations are sometimes used. Original California procedures required determination of the ratio at 0.1-in. intervals to 0.5 in. Corps of Engineers' procedures require determination of the ratio at 0.1 in. and 0.2 in. Where the ratio at 0.2 in. is consistently higher than at 0.1 in., the ratio at 0.2 in. is used.

41. CAPILLARY ACTION (CAPILLARITY)

The rise or movement of water in the interstices of a "soil" due to capillary forces.

42. CAPILLARY FLOW

See Capillary Migration.

43. CAPILLARY FRINGE ZONE

The zone above the "free water elevation" in which water is held by "capillary action."

44. CAPILLARY HEAD

h L

The potential, expressed in head of water, that causes the water to flow by "capillary action."

45. CAPILLARY MIGRATION (CAPILLARY FLOW)

The movement of water by "capillary action."

46. CAPILLARY RISE (HEIGHT OF CAPILLARY RISE)

h_c L

The height above a "free water elevation" to which water will rise by "capillary action."

47. CAPILLARY WATER

Water subject to the influence of "capillary action."

48. CENTRIFUGE MOISTURE EQUIVALENT

See Moisture Equivalent.

49. CLAY (CLAY SOIL)

Fine-grained "soil" or the fine-grained portion of "soil" that can be made to exhibit plasticity (putty-like properties) within a range of "water contents," and which exhibits considerable strength when air-dry. The term has been used to designate the percentage finer than 0.002 mm (0.005 in some cases), but it is strongly recommended that this usage be discontinued, since there is ample evidence that from an engineering standpoint the properties described in the above definition are many times more important.

50. CLAY SIZE

That portion of the "soil" finer than 0.002 mm (0.005 mm in some cases). (See discussion under Clay.)

51. CLAY SOIL

See Clay.

52. COBBLE (COBBLESTONE)

A "rock" fragment usually rounded or semirounded with an average dimension between 3 and 12 in.

53. COEFFICIENT OF ABSOLUTE VISCOSITY

See Coefficient of Viscosity.

54. COEFFICIENT OF ACTIVE EARTH PRESSURE

See Coefficient of Earth Pressure

*55. COEFFICIENT OF COMPRESSIBILITY (COEFFICIENT OF COMPRESSION)

a_v L^2F^{-1}

The secant slope, for a given pressure increment, of the "pressure-void ratio curve." Where a stress-strain curve is used, the slope of this curve is equal to $\frac{a_v}{1 + e}$

56. COEFFICIENT OF CONSOLIDATION

c_v L^2T^{-1}

A coefficient utilized in the theory of consolidation, containing the physical constants of a "soil" affecting its rate of volume change.

$c_v = \frac{k(1 + 3)}{a_v . \gamma w}$, wherein

k = "coefficient of permeability," LT^{-1}

e = "void ratio," D

a_v = "coefficient of compressibility," L^2F^{-1}

γ_w = unit weight of water, FL^{-3}

Note: In the literature published prior to 1935, the "coefficient of consolidation," usually designated c, was defined by the equation

$$c = \frac{k}{a_v \cdot \gamma_w (1 + e)}.$$

This original definition of the "coefficient of consolidation" may be found in some more recent papers and care should be taken to avoid confusion.

57. COEFFICIENT OF EARTH PRESSURE

K D

The "principal stress" ratio at a point in a "soil" mass.

58. ACTIVE

K_A D

The minimum ratio of (1) the "minor principal stress" to (2) the "major principal stress." This is applicable where the "soil" has yielded sufficiently to develop a lower limiting value of the "minor principal stress."

59. AT REST

K_0 D

The ratio of (1) the "minor principal stress" to (2) the "major principal stress." This is applicable where the "soil" mass is in its natural state without having been permitted to yield or without having been compressed.

60. PASSIVE

K_p D

The maximum ratio of (1) the "major principal stress" to (2) the "minor principal stress." This is applicable where the "soil" has been compressed sufficiently to develop an upper limiting value of the "major principal stress."

61. COEFFICIENT OF INTERNAL FRICTION

The tangent of the "angle of internal friction." (See Internal Friction.)

62. COEFFICIENT OF PERMEABILITY (PERMEABILITY)

k LT^{-1}

The rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature conditions (usually 20° C).

63. COEFFICIENT OF SUBGRADE REACTION (MODULUS OF SUBGRADE REACTION)

k FL^{-3}

Ratio of (1) load per unit area of horizontal surface of a mass of "soil" to (2) corresponding settlement of the surface. It is determined as the slope of the secant, drawn between the point corresponding to zero settlement and the point of 0.05-in. settlement, of a load settlement curve obtained from a plate load test on a "soil" using a 30-in. or greater diameter loading plate. It is used in the design of concrete pavements by the Westergaard Method.

64. COEFFICIENT OF UNIFORMITY

C_u D

The ratio D_{60}/D_{10}, where D_{60} is the particle diameter corresponding to 60 per cent finer on the grain-size curve, and D_{10} is the particle diameter corresponding to 10 per cent finer on the grain-size curve.

65. COEFFICIENT OF VISCOSITY (COEFFICIENT OF ABSOLUTE VISCOSITY)

μ FTL^{-2}

The shearing force per unit area required to maintain a unit difference in velocity between two parallel layers of a fluid a unit distance apart.

66. COEFFICIENT OF VOLUME COMPRESSIBILITY (MODULUS OF VOLUME CHANGE)

m_v L^2F^{-1}

The compression of a "soil" layer per unit of original thickness due to a given unit increase in pressure. It is numerically equal to the "coefficient of compressibility" divided by one plus the original "void ratio": $\frac{a_v}{1+e}$

*67. COHESION

c FL^{-2}

The portion of the "shear strength" of a "soil" indicated by the term c in Coulomb's equation, $s = c + p \tan \phi$.

68. APPARENT COHESION

Cohesion in granular "soils" due to capillary forces.

69. COHESIONLESS SOIL

A "soil" that when unconfined has little or no strength when air-dried, and that has little or no "cohesion" when submerged.

70. COHESIVE SOIL

A "soil" that when unconfined has considerable strength when air-dried, and that has significant "cohesion" when submerged.

71. COLLOIDAL PARTICLES

"Soil" particles that are so small that the surface activity has an appreciable influence on the properties of the aggregate.

72. COMPACTION

The densification of a "soil" by means of mechanical manipulation.

73. COMPACTION CURVE (PROCTOR CURVE) (MOISTURE-DENSITY CURVE)

The curve showing the relationship between the "dry unit weight" (density) and the "water content" of a "soil" for a given compactive effort.

74. COMPACTION TEST (MOISTURE-DENSITY TEST)

A laboratory compacting procedure whereby a "soil" at a known "water content" is placed in a specified manner into a mold of given dimensions, subjected to a compactive effort of controlled magnitude, and the resulting "unit weight" determined. The procedure is repeated for various "water contents" sufficient to establish a relation between "water content" and "unit weight."

75. COMPRESSIBILITY

Property of a "soil" pertaining to its susceptibility to decrease in volume when subjected to load.

76. COMPRESSION CURVE

See Pressure-Void Ratio Curve.

77. COMPRESSION INDEX

C_c D

The slope of the linear portion of the "pressure-void ratio curve" on a semi-log plot.

78. COMPRESSIVE STRENGTH (UNCONFINED COMPRESSIVE STRENGTH)

p_c, q_u FL^{-2}

The load per unit area at which an unconfined prismatic or cylindrical specimen of "soil" will fail in a simple compression test.

79. CONCENTRATION FACTOR

n D

A parameter used in modifying the Boussinesq equations to describe various distributions of vertical "stress."

80. CONSISTENCY

The relative ease with which a "soil" can be deformed.

81. CONSISTENCY INDEX

See Relative Consistency.

82. CONSOLIDATED DRAINED TEST (SLOW TEST)

A "soil" test in which essentially complete "consolidation" under the confining pressure is followed by additional axial (or shearing) "stress" applied in such a manner that even a fully saturated "soil" of low "permeability" can adapt itself completely (fully consolidate) to the changes in "stress" due to the additional axial (or shearing) "stress."

83. CONSOLIDATED UNDRAINED TEST (CONSOLIDATED QUICK TEST)

A "soil" test in which essentially complete "consolidation" under the vertical load (in a "direct shear test") or under the confining pressure (in a "triaxial test") is followed by a shear at constant "water content."

84. CONSOLIDATION

The gradual reduction in volume of a "soil" mass resulting from an increase in compressive "stress."

85. INITIAL CONSOLIDATION (INITIAL COMPRESSION)

A comparatively sudden reduction in volume of a "soil" mass under an applied load due principally to expulsion and compression of gas in the "soil" "voids" preceding "primary consolidation."

86. PRIMARY CONSOLIDATION (PRIMARY COMPRESSION) (PRIMARY TIME EFFECT)

The reduction in volume of a "soil" mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the "void" spaces of the mass and accompanied by a transfer of the load from the "soil" water to the "soil" solids.

87. SECONDARY CONSOLIDATION (SECONDARY COMPRESSION) (SECONDARY TIME EFFECT)

The reduction in volume of a "soil" mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal "structure" of the "soil" mass after most of the load has been transferred from the "soil" water to the "soil" solids.

88. CONSOLIDATION CURVE

See Consolidation Time Curve.

89. CONSOLIDATION RATIO

U_z D

The ratio of (1) the amount of "consolidation" at a given distance from a drainage surface and at a given time to (2) the total amount of "consolidation" obtainable at that point under a given "stress" increment.

90. CONSOLIDATION TEST

A test in which the specimen is laterally confined in a ring and is compressed between porous plates.

91. CONSOLIDATION-TIME CURVE (TIME CURVE) (CONSOLIDATION CURVE) (THEORETICAL TIME CURVE)

A curve that shows the relation between (1) the "degree of consolidation" and (2) the elapsed time after the application of a given increment of load.

92. CONTACT PRESSURE

p FL^{-2}

The unit of "pressure" that acts at the surface of contact between a structure and the underlying "soil" mass.

93. CONTROLLED-STRAIN TEST

A test in which the load is so applied that a controlled rate of "strain" results.

94. CONTROLLED-STRESS TEST

A test in which the "stress" to which a specimen is subjected is applied at a controlled rate.

95. CREEP

Slow movement of rock debris or soil usually imperceptible except to observations of long duration.

96. CRITICAL CIRCLE (CRITICAL SURFACE)

The sliding surface assumed in a theoretical analysis of a "soil" mass for which the factor of safety is a minimum.

97. CRITICAL DENSITY

The "unit weight" of a saturated granular material below which it will lose strength and above which it will gain strength when subjected to rapid deformation. The critical "density" of a given material is dependent on many factors.

98. CRITICAL HEIGHT

H_c L

The maximum height at which a vertical or sloped bank of "soil" will stand unsupported under a given set of conditions.

99. CRITICAL HYDRAULIC GRADIENT

See Hydraulic Gradient.

100. CRITICAL SLOPE

The maximum angle with the horizontal at which a sloped bank of "soil" of given height will stand unsupported.

101. CRITICAL SURFACE

See Critical Circle.

102. CRITICAL VOID RATIO

See Void Ratio.

103. CRYOLOGY

The study of the properties of snow, ice, and frozen ground.

104. DEFLOCCULATING AGENT (DEFLOCCULANT) (DISPERSING AGENT)

An agent that prevents fine "soil" particles in suspension from coalescing to form flocs.

105. DEFORMATION

Change in shape.

106. DEGREE OF CONSOLIDATION (PER CENT CONSOLIDATION)

U D

The ratio, expressed as a percentage, of (1) the amount of "consolidation" at a given time within a "soil" mass, to (2) the total amount of "consolidation" obtainable under a given "stress" condition.

107. DEGREE-DAYS

The difference between the average temperature each day and 32 F. In common usage degree-days are positive for daily average temperatures above 32 F and negative for those below 32 F. See also Freezing Index.

108. DEGREE OF SATURATION

See Per Cent Saturation.

109. DEGREE OF SENSITIVITY (SENSITIVITY RATIO)

See Remolding Index.

110. DENSITY

See Unit Weight.

Note: Although it is recognized that "density" is defined as mass per unit volume, in the field of "soil" mechanics the term is frequently used in place of unit weight.

111. DEVIATOR STRESS

Δ, σ FL^{-2}

The difference between the "major" and "minor principal stresses" in a "triaxial test."

112. DILATANCY

The expansion of cohesionless "soils" when subject to shearing deformation.

113. DIRECT SHEAR TEST

A shear test in which "soil" under an applied normal load is stressed to failure by moving one section of the "soil" container (shear box) relative to the other section.

114. DISCHARGE VELOCITY

v LT^{-1}

Rate of discharge of water through a porous medium per unit of total area perpendicular to the direction of flow.

115. DISPERSING AGENT

See Deflocculating Agent.

116. DRAWDOWN

L

Vertical distance the "free water elevation" is lowered or the reduction of the "pressure head" due to the removal of "free water."

117. DRY UNIT WEIGHT (DRY DENSITY)

See Unit Weight.

118. EARTH

See Soil.

119. EARTH PRESSURE

Unit: p	FL^{-2}
Total: P	F or FL^{-1}

The pressure or force exerted by "soil" on any boundary.

120. ACTIVE EARTH PRESSURE

P_A, p_A

The minimum value of "earth pressure." This condition exists when a "soil" mass is permitted to yield sufficiently to cause its internal "shearing" resistance along a potential failure surface to be completely mobilized.

121. AT REST

P_O, p_O

The value of the "earth pressure" when the "soil" mass is in its natural state without having been permitted to yield or without having been compressed.

122. PASSIVE EARTH PRESSURE

P_P, p_p

The maximum value of "earth pressure." This condition exists when a "soil" mass is compressed sufficiently to cause its internal "shearing" resistance along a potential failure surface to be completely mobilized.

123. EFFECTIVE DIAMETER (EFFECTIVE SIZE)

D_{10}, D_e — L

Particle diameter corresponding to 10 per cent finer on the grain-size curve.

124. EFFECTIVE DRAINAGE POROSITY

See Effective Porosity.

125. EFFECTIVE FORCE

$\overline{F}$ — F

The force transmitted through a "soil" mass by "intergranular pressures."

126. EFFECTIVE POROSITY (EFFECTIVE DRAINAGE POROSITY)

n_e — D

The ratio of (1) the volume of the "voids" of a "soil" mass that can be drained by gravity to (2) the total volume of the mass.

126A. EFFECTIVE PRESSURE

See Stress.

127. EFFECTIVE SIZE

See Effective Diameter.

128. EFFECTIVE STRESS

See Stress.

129. EFFECTIVE UNIT WEIGHT

See Unit Weight.

130. ELASTIC STATE OF EQUILIBRIUM

State of "stress" within a "soil" mass when the internal resistance of the mass is not fully mobilized.

131. EQUIPOTENTIAL LINE

Line along which water will rise to the same elevation in piezometric tubes.

132. EQUIVALENT DIAMETER (EQUIVALENT SIZE)

D L

The diameter of a hypothetical sphere composed of material having the same "specific gravity" as that of the actual "soil" particle and of such size that it will settle in a given liquid at the same terminal velocity as the actual "soil" particle.

133. EQUIVALENT FLUID

A hypothetical fluid having a "unit weight" such that it will produce a pressure against a lateral support presumed to be equivalent to that produced by the actual "soil." This simplified approach is valid only when deformation conditions are such that the pressure increases lineraly with depth and the "wall friction" is neglected.

134. EXCESS HYDROSTATIC PRESSURE

See Hydrostatic Pressure.

135. EXCHANGE CAPACITY

The capacity to exchange ions as measured by the quantity of exchangeable ions in a "soil."

136. FAILURE BY RUPTURE

See Shear Failure.

137. FIELD MOISTURE EQUIVALENT

See Moisture Equivalent.

138. FILL

Man-made deposits of natural "soils" and waste materials.

139. FILTER (PROTECTIVE FILTER)

A layer or combination of layers of pervious materials designed and installed in such a manner as to provide drainage, yet prevent the movement of "soil" particles due to flowing water.

140. FINES

Portion of a "soil" finer than a No. 200 U.S. standard sieve.

141. FLOC

Loose, open-structured mass formed in a suspension by the aggregation of minute particles.

142. FLOCCULATION

The process of forming flocs.

143. FLOCCULENT STRUCTURE

See Soil Structure.

144. FLOW CHANNEL

The portion of a "flow net" bounded by two adjacent "flow lines."

145. FLOW CURVE

The locus of points obtained from a standard "liquid limit" test and plotted on a graph representing "water content" as ordinate on an arithmetic scale and the number of blows as abscissa on a logarithmic scale.

146. FLOW FAILURE

Failure in which a "soil" mass moves over relatively long distances in a fluid-like manner.

147. FLOW INDEX

F_w, I_f D

The slope of the "flow curve" obtained from a "liquid limit" test, expressed as the difference in water contents at 10 blows and at 100 blows.

148. FLOW LINE

The path that a particle of water follows in its course of "seepage" under laminar flow conditions.

149. FLOW NET

A graphical representation of "flow lines" and "equipotential lines" used in the study of "seepage" phenomena.

150. FLOW SLIDE

The failure of a sloped bank of "soil" in which the movement of the "soil" mass does not take place along a well-defined surface of sliding.

151. FLOW VALUE

N_ϕ D

A quantity equal to $\tan^2(45° + \frac{\phi}{2})$

152. FOOTING

Portion of the "foundation" of a structure that transmits loads directly to the "soil."

153. FOUNDATION

Lower part of a structure that transmits the load to the earth.

154. FOUNDATION "SOIL"

Upper part of the earth mass carrying the load of the structure.

155. FREE WATER (GRAVITATIONAL WATER) (GROUND WATER) (PHREATIC WATER)

Water that is free to move through a "soil" mass under the influence of gravity.

156. FREE WATER ELEVATION (WATER TABLE) (GROUND WATER SURFACE) (FREE WATER SURFACE) (GROUND WATER ELEVATION)

Elevations at which the pressure in the water is zero with respect to the atmospheric pressure.

157. FREEZING INDEX

F "degree days"

The number of "degree days" between the highest and lowest points on the cumulative "degree days" - time curve for one freezing season. It is used as a measure of the combined duration and magnitude of below-freezing temperature occurring during any given freezing season. The index determined for air temperatures at 4.5 ft above the ground is commonly designated as the air freezing index, while that determined for temperatures immediately below a surface is known as the surface freezing index.

158. FROST ACTION

Freezing and thawing of moisture in materials and the resultant effects on these materials and on "structures" of which they are a part or with which they are in contact.

159. FROST BOIL

a. Softening of "soil" occurring during a thawing period due to the liberation of water from ice lenses or layers.

b. A hole formed in flexible pavements by the extrusion of soft "soil" and melt waters under the action of wheel loads.

c. Breaking of a highway or airfield pavement under traffic and the ejection of subgrade "soil" in a soft and soupy condition caused by the melting of ice lenses formed by "frost action."

160. FROST HEAVE

The raising of a surface due to the accumulation of ice in the underlying "soil."

161. GENERAL SHEAR FAILURE

See Shear Failure.

162. GLACIAL TILL (TILL)

Material deposited by glaciation, usually composed of a wide range of particle sizes, which has not been subjected to the sorting action of water.

163. GRADATION (GRAIN SIZE DISTRIBUTION) ("SOIL" TEXTURE)

Proportion of material of each grain size present in a given "soil."

164. GRAIN SIZE ANALYSIS (MECHANICAL ANALYSIS)

The process of determining "gradation."

165. GRAVEL

Rounded or semirounded particles of rock that will pass a 3-in. and be retained on a No. 4 U.S. standard sieve.

166. GRAVITATIONAL WATER

See Free Water.

167. GROUND WATER

See Free Water

168. GROUND WATER ELEVATION

See Free Water Elevation.

169. GROUND WATER SURFACE

See Free Water Elevation.

170. HARDPAN

Layer of extremely dense "soil."

171. HEAVE

Upward movement of "soil" caused by expansion or displacement resulting from phenomena such as: moisture absorption, removal of overburden, driving of piles, and "frost action."

172. HEIGHT OF CAPILLARY RISE

See Capillary Rise.

173. HOMOGENEOUS MASS

A mass that exhibits essentially the same physical properties at every point throughout the mass.

174. HONEYCOMB STRUCTURE

See Soil Structure.

175. HORIZON (SOIL HORIZON)

One of the layers of the "soil profile," distinguished principally by its texture, color, structure, and chemical content.

176. "A" HORIZON

The uppermost layer of a "soil profile" from which inorganic colloids and other soluble materials have been leached. Usually contains remnants of organic life.

177. "B" HORIZON

The layer of a "soil profile" in which material leached from the overlying "A horizon" is accumulated.

178. "C" HORIZON

Undisturbed "parent material" from which the overlying "soil profile" has been developed.

179. HUMUS

A brown or black material formed by the partial decomposition of vegetable or animal matter; the organic portion of "soil."

180. HYDRAULIC GRADIENT

i, s D

The loss of hydraulic head per unit distance of flow; $\frac{dh}{dL}$

181. CRITICAL HYDRAULIC GRADIENT

i_c D

Hydraulic gradient" at which the "intergranular pressure" in a mass of "cohesionless soil" is reduced to zero by the upward flow of water.

182. HYDROSTATIC PRESSURE

u_o FL^{-2}

The pressure in a liquid under static conditions; the product of the "unit weight" of the liquid and the difference in elevation between the given point and the "free water elevation."

183. EXCESS HYDROSTATIC PRESSURE (HYDROSTATIC EXCESS PRESSURE)

$\bar{u}$, u FL^{-2}

The pressure that exists in pore water in excess of the "hydrostatic pressure."

184. HYGROSCOPIC CAPACITY (HYGROSCOPIC COEFFICIENT)

w_c D

Ratio of (1) the weight of water absorbed by a dry "soil" in a saturated atmosphere at a given temperature to (2) the weight of the oven-dried "soil."

185. HYGROSCOPIC WATER CONTENT

w_H D

The "water content" of an air-dried "soil."

186. INFLUENCE VALUE

I D

The value of the portion of a mathematical expression that contains combinations of the independent variables arranged in dimensionless form.

187. INITIAL CONSOLIDATION (INITIAL COMPRESSION)

See Consolidation.

188. INORGANIC SILT

See Silt.

189. INTERGRANULAR PRESSURE

See Stress.

190. INTERMEDIATE PRINCIPAL PLANE

See Principal Plane.

191. INTERMEDIATE PRINCIPAL STRESS

See Stress.

192. INTERNAL FRICTION

FL^{-2}

The portion of the "shearing strength" of a "soil" indicated by the terms p tan ϕ in Coulomb's equation $s = c + p \tan \phi$. It is usually considered to be due to the interlocking of the "soil" grains and the resistance to sliding between the grains.

193. ISOCHRONE

A curve showing the distribution of the "excess hydrostatic pressure" at a given time during a process of "consolidation."

194. ISOTROPIC MASS

A mass having the same property (or properties) in all directions.

195. KAOLIN

A variety of clay containing a high percentage of kaolinite.

*196. LAMINAR FLOW (STREAMLINE FLOW) (VISCOUS FLOW)

Flow in which the head loss is proportional to the first power of the velocity.

*197. LANDSLIDE (SLIDE)

The failure of a sloped bank of "soil" in which the movement of the "soil" mass takes place along a surface of sliding.

198. LEACHING

The removal of soluble "soil" material and colloids by percolating water.

199. LEDGE

See Bed Rock.

200. LINE OF CREEP (PATH OF PERCOLATION)

The path that water follows along the impervious surface of contact between the foundation soil and the base of a dam or other structure.

201. LINE OF SEEPAGE (SEEPAGE LINE) (PHREATIC LINE)

The upper "free water surface" of the zone of "seepage."

202. LINEAR EXPANSION

L_E D

The increase in one dimension of a "soil" mass, expressed as a percentage of that dimension at the "shrinkage limit," when the "water content" is increased from the "shrinkage limit" to any given "water content."

203. LINEAR SHRINKAGE

L_S D

Decrease in one dimension of a "soil" mass, expressed as a percentage of the original dimension, when the "water content" is reduced from a given value to the "shrinkage limit."

204. LIQUID LIMIT

LL, L_w, w_L D

a. The "water content" corresponding to the arbitrary limit between the liquid and plastic states of consistency of a "soil."

b. The "water content" at which a pat of "soil," cut by a groove of standard dimensions, will flow together for a distance of 1/2 in. under the impact of 25 blows in a standard liquid limit apparatus.

205. LIQUEFACTION (SPONTANEOUS LIQUEFACTION)

The sudden large decrease of the shearing resistance of a "cohesionless soil." It is caused by a collapse of the "structure" by shock or other type of "strain" and is associated with a sudden but temporary increase of the porefluid pressure. It involves a temporary transformation of the material into a fluid mass.

206. LIQUIDITY INDEX (WATER-PLASTICITY RATIO) (RELATIVE WATER CONTENT)

B, R_w, I_L D

The ratio, expressed as a percentage, of (1) the natural "water content" of a "soil" minus its "plastic limit" to (2) its "plasticity index."

207. LOAM

A mixture of "sand," "silt," or "clay," or a combination of any of these, with organic matter—"humus." It is sometimes called "topsoil" in contrast to the "subsoils" that contain little or no organic matter.

208. LOCAL SHEAR FAILURE

See Shear Failure.

209. LOESS

A uniform "aeolian deposit" of silty material having an open structure and relatively high cohesion due to cementation of "clay" or calcareous material at grain contacts. A characteristic of loess deposits is that they can stand with nearly vertical slopes.

210. MAJOR PRINCIPAL PLANE

See Principal Plane.

211. MAJOR PRINCIPAL STRESS

See Stress.

212. MASS UNIT WEIGHT

See Unit Weight.

213. MAXIMUM DENSITY (MAXIMUM UNIT WEIGHT)

See Unit Weight.

214. MECHANICAL ANALYSIS

See Grain Size Analysis

215. MINOR PRINCIPAL PLANE

See Principal Plane

216. MINOR PRINCIPAL STRESS

See Stress.

217. MODULUS OF DEFORMATION

See Modulus of Elasticity

218. MODULUS OF ELASTICITY (MODULUS OF DEFORMATION)

E, M FL^{-2}

The ratio of "stress" to "strain" for a material under given loading conditions; numerically equal to the slope of the tangent or the secant of a stress-strain curve. The use of the term "Modulus of Elasticity" is recommended for materials that deform in accordance with Hooke's Law; the term "Modulus of Deformation" for materials that deform otherwise.

219. MODULUS OF SUBGRADE REACTION

See Coefficient of Subgrade Reaction.

220. MODULUS OF VOLUME CHANGE

See Coefficient of Volume Compressibility.

221. MOHR CIRCLE

A graphical representation of the stresses acting on the various planes at a given point.

*222. MOHR ENVELOPE (rupture envelope) (RUPTURE LINE)

The envelope of a series of "Mohr Circles" representing "stress" conditions at failure for a given material. According to Mohr's rupture hypothesis, a rupture envelope is the locus of points the coordinates of which represent the combinations of "normal and shearing stresses" that will cause a given material to fail.

223. MOISTURE CONTENT (WATER CONTENT)

w D

The ratio, expressed as a percentage, of (1) the weight of water in a given "soil" mass to (2) the weight of solid particles.

224. MOISTURE DENSITY CURVE

See Compaction Curve.

225. MOISTURE DENSITY TEST

See Compaction Test.

226. MOISTURE EQUIVALENT

CENTRIFUGE MOISTURE EQUIVALENT

W_c, CME D

The "water content" of a "soil" after it has been saturated with water and then subjected for one hour to a force equal to 1000 times that of gravity.

227. FIELD MOISTURE EQUIVALENT

FME

The minimum "water content," expressed as a percentage of the weight of the oven-dried "soil," at which a drop of water placed on a smoothed surface of the "soil" will not immediately be absorbed by the "soil" but will spread out over the surface and give it a shiny appearance.

228. MUCK

An organic "soil" of very soft consistency.

229. MUD

A mixture of "soil" and water in a fluid or weakly solid state.

230. MUSKEG

Level, practically treeless areas supporting dense growth consisting primarily of grasses. The surface of the "soil" is covered with a layer of partially decayed grass and grass roots which is usually wet and soft when not frozen.

231. NEUTRAL STRESS

See Stress.

232. NORMAL STRESS

See Stress.

233. NORMALLY CONSOLIDATED "SOIL" DEPOSIT

A soil deposit that has never been subjected to an "effective pressure" greater than the existing overburden pressure and one that is also completely consolidated by the existing overburden.

234. OPTIMUM MOISTURE CONTENT (OPTIMUM WATER CONTENT)

OMC, W_o D

The "water content" at which a "soil" can be compacted to the maximum "dry unit weight" by a given compactive effort.

235. ORGANIC CLAY

A "clay" with a high organic content.

236. ORGANIC SILT

A "silt" with a high organic content.

237. ORGANIC SOIL

"Soil" with a high organic content. In general, organic "soils" are very compressible and have poor load-sustaining properties.

*238. OVERCONSOLIDATED SOIL DEPOSIT

A "soil" deposit that has been subjected to an "effective pressure" greater than the present overburden pressure.

239. PARENT MATERIAL

Material from which a "soil" has been derived.

240. PASSIVE EARTH PRESSURE

See Earth Pressure.

241. PASSIVE STATE OF PLASTIC EQUILIBRIUM

See Plastic Equilibrium.

242. PATH OF PERCOLATION

See Line of Creep

243. PAVEMENT PUMPING

Ejection of "soil" and water mixtures from joints cracks and edges of rigid pavements, under the action of traffic.

244. PEAT

A fibrous mass of "organic" matter in various stages of decomposition, generally dark brown to black in color and of spongy consistency.

245. PENETRATION RESISTANCE (STANDARD PENETRATION RESISTANCE) (PROCTOR PENETRATION RESISTANCE)

p_R, N $\quad$ FL^{-2} or Blows L^{-1}

a. Number of blows of a hammer of specified weight falling a given distance required to produce a given penetration into "soil" of a pile, casing, or sampling tube.

b. Unit load required to maintain constant rate of penetration into "soil" of a probe or instrument.

c. Unit load required to produce a specified penetration into "soil" at a specified rate of a probe or instrument. For a Proctor needle, the specified penetration is 2-1/2 in. and the rate is 1/2 in. per second.

246. PENETRATION RESISTANCE CURVE (PROCTOR PENETRATION CURVE)

The curve showing the relationship between (1) the "penetration resistance" and (2) the "water content."

247. PER CENT COMPACTION

The ratio, expressed as a percentage, of (1) "dry unit weight" of a "soil" to (2) "maximum unit weight" obtained in a laboratory "compaction test."

248. PER CENT CONSOLIDATION

See Degree of Consolidation.

249. PER CENT SATURATION (DEGREE OF SATURATION)

S D

The ratio, expressed as a percentage, of (1) the volume of water in a given "soil" mass to (2) the total volume of intergranular space ("voids").

250. PERCHED WATER TABLE

A water table usually of limited area maintained above the normal "free water elevation" by the presence of an intervening relatively impervious confining strata.

251. PERCOLATION

The movement of "gravitational water" through "soil." (See Seepage)

252. PERMAFROST

Perennially frozen "soil."

253. PERMEABILITY

See Coefficient of Permeability.

254. pH

pH D

An index of the acidity or alkalinity of a "soil" in terms of the logarithm of the reciprocal of the hydrogen ion concentration.

255. PHREATIC LINE

See Line of Seepage.

256. PHREATIC SURFACE

See Free Water Elevation.

257. PHREATIC WATER

See Free Water.

258. PIEZOMETER

An instrument for measuring pressure head.

259. PIEZOMETRIC SURFACE

The surface at which water will stand in a series of "piezometers."

260. PILE

Relatively slender structural element which is driven, or otherwise introduced, into the "soil", usually for the purpose of providing vertical or lateral support.

261. PIPING

The movement of "soil" particles by percolating water leading to the development of channels.

262. PLASTIC DEFORMATION

See Plastic Flow.

263. PLASTIC EQUILIBRIUM

State of "stress" within a "soil" mass or a portion thereof, which has been deformed to such an extent that its ultimate shearing resistance is mobilized.

264. ACTIVE STATE OF PLASTIC EQUILIBRIUM

"Plastic equilibrium" obtained by an expansion of a mass.

265. PASSIVE STATE OF PLASTIC EQUILIBRIUM

"Plastic equilibrium" obtained by a compression of a mass.

266. PLASTICITY

The property of a "soil" which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change.

267. PLASTIC FLOW (PLASTIC DEFORMATION)

The "deformation" of a plastic material beyond the point of recovery, accompanied by continuing "deformation" with no further increase in stress.

268. PLASTIC LIMIT

w_p, PL, P_w D

a. The "water content" corresponding to an arbitrary limit between the plastic and the semisolid states of "consistency" of a soil.

b. "Water content" at which a "soil" will just begin to crumble when rolled into a thread approximately 1/8 in. in diameter.

269. PLASTIC SOIL

A "soil" that exhibits "plasticity."

270. PLASTIC STATE (PLASTIC RANGE)

The range of consistency within which a "soil" exhibits plastic properties.

271. PLASTICITY INDEX

I_p, PI, I_w D

Numerical difference between the "liquid limit" and the "plastic limit."

272. PORE PRESSURE (PORE WATER PRESSURE)

See "Neutral Stress" under Stress.

273. POROSITY

n D

The ratio, usually expressed as a percentage, of (1) the volume of "voids" of a given "soil" mass to (2) the total volume of the "soil" mass.

*274. POTENTIAL DROP

Δh L

The difference in "pressure" head between two "equipotential lines."

275. PRECONSOLIDATION PRESSURE (PRESTRESS)

p_c FL^{-2}

The greatest "effective pressure" to which a "soil" has been subjected.

276. PRESSURE

p FL^{-2}

The load divided by the area over which it acts.

277. PRESSURE BULB

The zone in a loaded "soil" mass bounded by an arbitrarily selected isobar of "stress."

278. PRESSURE - VOID RATIO CURVE (COMPRESSION CURVE)

A curve representing the relationship between "effective pressure" and "void ratio" of a "soil" as obtained from a "consolidation test." The curve has a characteristic shape when plotted on semilog paper with "effective pressure" on the log scale. The various parts of the curve and extensions to the parts have been designated as recompression, compression, virgin compression, expansion, rebound, and other descriptive names by various authorities.

279. PRIMARY CONSOLIDATION (PRIMARY COMPRESSION) (PRIMARY TIME EFFECT)

See Consolidation.

280. PRINCIPAL PLANE

Each of three mutually perpendicular planes through a point in a "soil" mass on which the "shearing stress" is zero.

281. INTERMEDIATE PRINCIPAL PLANE

The plane normal to the direction of the "intermediate principal stress."

282. MAJOR PRINCIPAL PLANE

The plane normal to the direction of the "major principal stress."

283. MINOR PRINCIPAL PLANE

The plane normal to the direction of the "minor principal stress."

284. PRINCIPAL STRESS

See Stress.

285. PROCTOR COMPACTION CURVE

See Compaction Curve.

286. PROCTOR PENETRATION CURVE

See Penetration Resistance Curve.

287. PROCTOR PENETRATION RESISTANCE

See Penetration Resistance.

288. PROFILE

See Soil Profile.

289. PROGRESSIVE FAILURE

Failure in which the ultimate shearing resistance is progressively mobilized along the failure surface.

290. PROTECTIVE FILTER

See Filter.

291. PUMPING OF PAVEMENT (PUMPING)

See Pavement Pumping.

292. QUICK CONDITION (QUICKSAND)

Condition in which water is flowing upwards with sufficient velocity to reduce significantly the "bearing capacity" of the "soil" through a decrease in "intergranular pressure."

293. QUICK TEST

See Unconsolidated Undrained Test.

294. RADIUS OF INFLUENCE OF A WELL

Distance from the center of the well to the closest point at which the piezometric surface is not lowered when pumping has produced the maximum steady rate of flow.

295. RELATIVE CONSISTENCY

I_c, C_r D

Ratio of (1) the "liquid limit" minus the natural "water content" to (2) the "plasticity index."

296. RELATIVE DENSITY

D_d D

The ratio of (1) the difference between the "void ratio" of a "cohesionless soil" in the loosest state and any given "void ratio" to (2) the difference between its "void ratios" in the loosest and in the densest states.

297. RELATIVE WATER CONTENT

See Liquidity Index.

298. REMOLDED SOIL

"Soil" that has had its natural structure modified by manipulation.

299. REMOLDING INDEX

I_R D

The ratio of (1) the "modulus of deformation" of a "soil" in the undisturbed state to (2) the "modulus of deformation" of the "soil" in the remolded state.

300. REMOLDING SENSITIVITY (SENSITIVITY RATIO)

S_t D

The ratio of (1) the "unconfined compressive strength" of an undisturbed specimen of "soil" to (2) the "unconfined compressive strength" of a specimen of the same "soil" after remolding at unaltered "water content."

301. RESIDUAL SOIL

"Soil" derived in place by weathering of the underlying material.

302. ROCK

Natural solid mineral matter occurring in large masses or fragments.

303. ROCK FLOUR

See Silt.

304. RUPTURE ENVELOPE (RUPTURE LINE)

See Mohr Envelope

305. SAND

Particles of rock that will pass the No. 4 sieve and be retained on the No. 200 U.S. standard sieve.

306. SAND BOIL

The ejection of sand and water resulting from "piping."

307. SATURATED UNIT WEIGHT

See Unit Weight.

308. SATURATION CURVE

See Zero Air Voids Curve.

309. SECONDARY CONSOLIDATION (SECONDARY COMPRESSION) (SECONDARY TIME EFFECT)

See Consolidation.

310. SEEPAGE (PERCOLATION)

The slow movement of "gravitational water" through the "soil."

311. SEEPAGE FORCE

J F

The force transmitted to the "soil" grains by "seepage."

312. SEEPAGE LINE

See Line of Seepage

313. SEEPAGE VELOCITY

v_s, v_1 LT^{-1}

The rate of discharge of seepage water through a porous medium per unit area of void space perpendicular to the direction of flow.

314. SENSITIVITY

The effect of remolding on the consistency of a "cohesive soil."

315. SHAKING TEST

A test used to indicate the presence of significant amounts of "rock flour," "silt," or very fine "sand" in a fine-grained "soil." It consists of shaking a pat of wet "soil," having a consistency of thick paste, in the palm of the hand; observing the surface for a glossy or livery appearance; then squeezing the pat; and observing if a rapid apparent drying and subsequent cracking of the "soil" occurs.

316. SHEAR FAILURE (FAILURE BY RUPTURE)

Failure in which movement caused by "shearing stresses" in a "soil" mass is of sufficient magnitude to destroy or seriously endanger a "structure."

317. GENERAL SHEAR FAILURE

Failure in which the ultimate strength of the "soil" is mobilized along the entire potential surface of sliding before the structure supported by the "soil" is impaired by excessive movement.

318. LOCAL SHEAR FAILURE

Failure in which the ultimate "shearing strength" of the "soil" is mobilized only locally along the potential surface of sliding at the time the structure supported by the "soil" is impaired by excessive movement.

319. SHEAR STRENGTH

s FL^{-2}

The maximum resistance of a "soil" to "shearing stresses."

320. SHEAR STRESS (SHEARING STRESS) (TANGENTIAL STRESS)

See Stress.

321. SHRINKAGE INDEX

SI D

The numerical difference between the "plastic and shrinkage limits."

322. SHRINKAGE LIMIT

SL D

The maximum "water content" at which a reduction in "water content" will not cause a decrease in volume of the "soil" mass.

323. SHRINKAGE RATIO

R D

The ratio of (1) a given volume change, expressed as a percentage of the dry volume, to (2) the corresponding change in "water content" above the "shrinkage limit," expressed as a percentage of the weight of the oven-dried "soil."

324. SILT (INORGANIC SILT) (ROCK FLOUR)

Material passing the No. 200 U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air-dried.

325. SILT SIZE

That portion of the soil finer than 0.02 mm and coarser than 0.002 mm (0.05 mm and 0.005 mm in some cases).

326. SINGLE-GRAINED STRUCTURE

See Soil Structure.

327. SKIN FRICTION

f FL^{-2}

The frictional resistance developed between "soil" and a "structure."

328. SLAKING

The process of breaking up or sloughing when an indurated "soil" is immersed in water.

329. SLOW TEST

See Consolidated-Drained Test.

330. SOIL (EARTH)

Sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter.

331. SOIL BINDER

See Binder

332. SOIL-FORMING FACTORS

Factors, such as parent material, climate, vegetation, topography, organisms, and time involved in the transformation of an original geologic deposit into a "soil profile."

333. SOIL HORIZON

See Horizon.

334. SOIL MECHANICS

The application of the laws and principles of mechanics and hydraulics to engineering problems dealing with "soil" as an engineering material.

335. SOIL PHYSICS

The organized body of knowledge concerned with the physical characteristics of "soil" and with the methods employed in their determinations.

336. SOIL PROFILE (PROFILE)

Vertical section of a soil, showing the nature and sequence of the various layers, as developed by deposition or weathering, or both.

337. SOIL STABILIZATION

Chemical or mechanical treatment designed to increase or maintain the "stability" of a mass of "soil" or otherwise to improve its engineering properties.

*338. SOIL STRUCTURE

The arrangement and state of aggregation of "soil" particles in a "soil" mass.

339. FLOCCULENT STRUCTURE

An arrangement composed of "flocs" of "soil" particles instead of individual "soil" particles.

340. HONEYCOMB STRUCTURE

An arrangement of "soil" particles having a comparatively loose, stable "structure" resembling a honeycomb.

341. SINGLE-GRAINED STRUCTURE

An arrangement composed of individual "soil" particles; characteristic structure of coarse-grained "soils."

342. SOIL SUSPENSION

Highly diffused mixture of "soil" and water.

343. SOIL TEXTURE

See Gradation.

344. SPECIFIC GRAVITY

345. SPECIFIC GRAVITY OF SOLIDS

G_s, S_s D

Ratio of (1) the weight in air of a given volume of "soil" solids at a stated temperature to (2) the weight in air of an equal volume of distilled water at a stated temperature.

346. APPARENT SPECIFIC GRAVITY

G_a, S_a D

Ratio of (1) the weight in air of a given volume of the impermeable portion of a permeable material (that is the solid matter including its impermeable pores or voids) at a stated temperature to (2) the weight in air of an equal volume of distilled water at a stated temperature.

347. BULK SPECIFIC GRAVITY (SPECIFIC MASS GRAVITY)

G_m, S_m D

Ratio of (1) the weight in air of a given volume of permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to (2) the weight in air of an equal volume of distilled water at a stated temperature.

348. SPECIFIC SURFACE

L^{-1}

The surface area per unit of volume of "soil" particles.

349. STABILITY FACTOR (STABILITY NUMBER)

N_s D

A pure number used in the analysis of the stability of a "soil" embankment. Defined by the following equation:

$$N_s = \frac{H_c \cdot \gamma_e}{c}$$

Where: H_c is critical height of the sloped bank
γ_e is the effective unit weight of the "soil"
c is the cohesion of the "soil"

Note: Taylor's "stability number" is the reciprocal of Terzaghi's "stability factor."

350. STABILIZATION

See Soil Stabilization.

351. STANDARD COMPACTION

See Compaction Test.

352. STANDARD PENETRATION RESISTANCE

See Penetration Resistance.

353. STICKY LIMIT

T_w D

The lowest "water content" at which a "soil" will stick to a metal blade drawn across the surface of the "soil" mass.

354. STONE

Crushed or naturally angular particles of "rock" that will pass a 3-in. sieve and be retained on a No. 4 U.S. standard sieve.

355. STRAIN

ϵ D

The change in length per unit of length in a given direction.

356. STREAMLINE FLOW

See Laminar Flow.

357. STRESS

σ, p, f FL^{-2}

The force per unit area acting within the "soil" mass.

358. EFFECTIVE STRESS (EFFECTIVE PRESSURE) (INTERGRANULAR PRESSURE)

$\bar{\sigma}$, $\bar{f}$ FL^{-2}

The average normal force per unit area transmitted from grain to grain of a "soil" mass. It is the "stress" that is effective in mobilizing "internal friction."

359. NEUTRAL STRESS (PORE PRESSURE) (PORE WATER PRESSURE)

u, u_w FL^{-2}

Stress transmitted through the pore water (water filling the voids of the soil).

360. NORMAL STRESS

σ, p FL^{-2}

The "stress" component normal to a given plane.

361. PRINCIPAL STRESS

σ_1, σ_2, σ_3 FL^{-2}

Stresses acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shearing stress is zero.

362. MAJOR PRINCIPAL STRESS

σ_1 FL^{-2}

The largest (with regard to sign) "principal stress."

363. MINOR PRINCIPAL STRESS

σ_3 FL^{-2}

The smallest (with regard to sign) "principal stress."

364. INTERMEDIATE PRINCIPAL STRESS

σ_2 FL^{-2}

The "principal stress" whose value is neither the largest nor the smallest (with regard to sign) of the three.

365. SHEAR STRESS (SHEARING STRESS) (TANGENTIAL STRESS)

τ, s FL^{-2}

The "stress" component tangential to a given plane.

366. TOTAL STRESS

σ, f FL^{-2}

The total force per unit area acting within a mass of "soil." It is the sum of the "neutral" and "effective stresses."

367. STRUCTURE

See Soil Structure.

368. SUBBASE

A layer used in a pavement system between the "subgrade" and "base course," or between the "subgrade" and portland-cement-concrete pavement.

369. SUBGRADE

The "soil" prepared and compacted to support a structure or a pavement system.

370. SUBGRADE SURFACE

The surface of the earth or "rock" prepared to support a structure or a pavement system.

371. SUBMERGED UNIT WEIGHT

See Unit Weight.

372. SUBSOIL

a. "Soil" below a "subgrade" or "fill."

b. That part of a "soil profile" occurring below the "'A' horizon."

373. TALUS

"Rock" fragments mixed with "soil" at the foot of a natural slope from which they have been separated.

374. TANGENTIAL STRESS

See Stress.

375. THEORETICAL TIME CURVE

See Consolidation Time Curve.

376. THERMO-OSMOSIS

The process by which water is caused to flow in small openings of a "soil" mass due to differences in temperature within the mass.

377. THIXOTROPY

The property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible.

378. TILL

See Glacial Till.

379. TIME CURVE

See Consolidation Time Curve.

380. TIME FACTOR

T_v, T D

Dimensionless factor, utilized in the theory of consolidation, containing the physical constants of a "soil" stratum influencing its time-rate of "consolidation," expressed as follows:

$$T = \frac{k\ (1 + e)t}{a_v\ \gamma_w \cdot H^2} = \frac{c_v\ .\ t}{H^2}$$

Where k = "coefficient of permeability" (LT^{-1})
e = "void ratio" (dimensionless)
t = elapsed time that the stratum has been consolidated (T)
a_v = "coefficient of compressibility" (L^2F^{-1})
γ_w = "unit weight of water" (FL^{-3})
H = thickness of stratum drained on one side only. If stratum is drained on both sides, its thickness equals 2H (L).
c_v = "coefficient of consolidation" (L^2T^{-1})

381. TOPSOIL

Surface "soil" usually containing organic matter.

382. TORSIONAL SHEAR TEST

A shear test in which a relatively thin test specimen of solid circular or annular cross section, usually confined between rings, is subjected to an axial load and to shear in torsion. Inplace torsion shear tests may be performed by pressing a dentated solid circular or annular plate against the "soil" and measuring its resistance to rotation under a given axial load.

383. TOTAL STRESS

See Stress.

384. TOUGHNESS INDEX

I_T, T_w

385. TRANSFORMED FLOW NET

A "flow net" whose boundaries have been properly modified (transformed) so that a net consisting of curvilinear squares can be constructed to represent flow conditions in an anisotropic porous medium.

386. TRANSPORTED SOIL

"Soil" transported from the place of its origin by wind, water, or ice.

387. TRIAXIAL SHEAR TEST (TRIAXIAL COMPRESSION TEST)

A test in which a cylindrical specimen of "soil" encased in an impervious membrane is subjected to a confining pressure and then loaded axially to failure.

388. TURBULENT FLOW

That type of flow in which any water particle may move in any direction with respect to any other particle, and in which the head loss is approximately proportional to the second power of the velocity.

389. ULTIMATE BEARING CAPACITY

q_o, q_{ult}, FL^{-2}

The average load per unit of area required to produce failure by rupture of a supporting "soil" mass.

390. UNCONFINED COMPRESSIVE STRENGTH

See Compressive Strength.

391. UNCONSOLIDATED-UNDRAINED TEST (QUICK TEST)

A "soil" test in which the "water content" of the test specimen remains practically unchanged during the application of the confining pressure and the additional axial (or shearing) force.

392. UNDERCONSOLIDATED SOIL DEPOSIT

A deposit that is not fully consolidated under the existing overburden pressure.

393. UNDISTURBED SAMPLE

A "soil" sample that has been obtained by methods in which every precaution has been taken to minimize disturbance to the sample.

394. UNIT WEIGHT

γ FL^{-3}

395. DRY UNIT WEIGHT (UNIT DRY WEIGHT)

γ_d, γ_o FL^{-3}

The weight of "soil" solids per unit of total volume of "soil" mass.

396. EFFECTIVE UNIT WEIGHT

γ_e FL^{-3}

That "unit weight" of a "soil" which, when multiplied by the height of the overlying column of "soil," yields the "effective pressure" due to the weight of the overburden.

397. MAXIMUM UNIT WEIGHT

$\gamma_{max.}$ FL^{-3}

The "dry unit weight" defined by the peak of a "compaction curve."

398. SATURATED UNIT WEIGHT

γ_G, $\gamma_{sat.}$ FL^{-3}

The "wet unit weight" of a "soil" mass when saturated.

399. SUBMERGED UNIT WEIGHT (BUOYANT UNIT WEIGHT)

γ_m, γ', $\gamma_{sub.}$ FL^{-3}

The weight of the solids in air minus the weight of water displaced by the solids per unit of volume of "soil" mass; the "saturated unit weight" minus the "unit weight of water."

400. UNIT WEIGHT OF WATER

γ_w FL^{-3}

The weight per unit volume of water; nominally equal to 62.4 lb per cu ft or 1 g per cu cm.

401. WET UNIT WEIGHT (MASS UNIT WEIGHT)

γ_m, $\gamma_{wet.}$ FL^{-3}

The weight (solids plus water) per unit of total volume of "soil" mass, irrespective of the "degree of saturation."

402. ZERO AIR VOIDS UNIT WEIGHT

γ_z FL^{-3}

The weight of solids per unit volume of a saturated "soil" mass.

403. UPLIFT

Unit: u FL^{-2}
Total: U F or FL^{-1}

The upward water pressure on a structure.

404. VANE SHEAR TEST

An in-place shear test in which a rod with thin radial vanes at the end is forced into the soil and the resistance to rotation of the rod is determined.

405. VARVED CLAY

Alternating thin layers of "silt" (or fine "sand") and "clay" formed by variations in sedimentation during the various seasons of the year, often exhibiting contrasting colors when partially dried.

406. VIRGIN COMPRESSION CURVE

See Compression Curve.

407. VISCOUS FLOW

See Laminar Flow.

408. VOID

Space in a "soil" mass not occupied by solid mineral matter. This space may be occupied by air, water, or other gaseous or liquid material.

409. VOID RATIO

e D

The ratio of (1) the volume of "void" space to (2) the volume of solid particles in a given "soil" mass.

410. CRITICAL VOID RATIO

e_c D

The "void ratio" corresponding to the "critical density."

411. VOLUMETRIC SHRINKAGE (VOLUMETRIC CHANGE)

V_s D

The decrease in volume, expressed as a percentage of the "soil" mass when dried, of a "soil" mass when the water content is reduced from a given percentage to the "shrinkage limit."

412. WALL FRICTION

f' FL^{-2}

Frictional resistance mobilized between a wall and the "soil" in contact with the wall.

413. WATER CONTENT

See Moisture Content.

414. WATER-HOLDING CAPACITY

D

The smallest value to which the "water content" of a "soil" can be reduced by gravity drainage.

415. WATER-PLASTICITY RATIO (RELATIVE WATER CONTENT) (LIQUIDITY INDEX)

See Liquidity Index

416. WATER TABLE

See Free Water Elevation.

417. WET UNIT WEIGHT

See Unit Weight.

418. ZERO AIR VOIDS CURVE (SATURATION CURVE)

The curve showing the "zero air voids unit weight" as a function of "water content."

419. ZERO AIR VOIDS DENSITY (ZERO AIR VOIDS UNIT WEIGHT)

See Unit Weight

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SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1828)

(Over)

DETERMINATION OF THE 0.02 MM FRACTION IN GRANULAR SOILS[a]

Closure by R. W. Johnson

R. W. JOHNSON,[1] A. M. ASCE.—The writer appreciates the noteworthy discussion and complementary analysis presented by Messrs. Fang and Sherman.

While a statistical method provides a means whereby a mass of data may be analyzed to obtain probable values, it is emphasized that it is not a means of alleviating the bulk of laboratory soils testing. Not only the heterogeneous characteristics of the soil itself, but also the human factor involved in sampling, testing, and evaluating results can lead to errors in the resultant data. It is reemphasized that the reported constant is valid only for soils having similar physical properties. In applying statistical method to the analysis of data, mathematical accuracies should not be ascribed which are greater than the presumed accuracies of the data being analyzed. This is particularly true of soils data.

Mr. Fang has presented data and derived statistical formulas for the amounts of material finer than 0.02, 0.005, and 0.002 mm for a group of soils having definite plastic properties. Formulas of this type have a significant value for the soils engineer working with soils whose properties are generally known, by permitting a rapid approximation of the amounts of finer fractions without employing lengthy laboratory tests. It can be expected that similar investigations will be conducted in the future with the possibility of eventually developing a family of formulas encompassing the entire range of engineering soils.

a. Proc. Paper 1309, July, 1957, by R. W. Johnson.
1. Eng. Div., USAREUR Const. Agency, Germany.

THIXOTROPIC CHARACTERISTICS OF COMPACTED CLAYS[a]

Closure by H. B. Seed and C. K. Chan

H. B. SEED,[1] A. M. ASCE and C. K. CHAN,[2] J. M. ASCE.—Mr. Barber points out that "Some thixotropy is caused by redistribution of non-uniformly distributed moisture to make a more uniform and therefore stronger material". The writers would not consider such a strength increase as thixotropy though moisture redistribution after compaction may well be a factor promoting strength increase in cases where the water and soil were poorly mixed initially. However, this is not believed to be a significant factor in the tests reported in the paper, for which the soil and water were well mixed and subsequently cured for one day to allow further moisture redistribution before compaction.

Furthermore, in contrast to the data presented by Mr. Barber, the writers' studies indicate that samples prepared by static compaction may exhibit appreciable thixotropic properties. Although the thixotropic strength increases for samples prepared by static compaction appear to be proportionately less than for samples prepared by kneading compaction, it is believed that this might well be due to the higher initial strength of samples prepared by static compaction. Mr. Barber's tests were conducted on samples prepared "near optimum moisture and maximum density". In the writers' experience, samples compacted to such a condition can show marked differences in strength due to minor changes in density and water content and, in the absence of this information, it is difficult to determine what proportion of the strength change reported by Mr. Barber may be due to composition changes and what proportion is due to other factors including thixotropy. Near optimum water content one would expect a relatively small thixotropic strength increase.

The writers welcome the historical background provided by Professor Krynine and agree with his remarks concerning the misleading conclusions which may be indicated by the use of the term "thixotropic strength ratio". While this may be a useful term in practice, it does not necessarily reflect the actual thixotropic strength increase in a soil. The writers cannot agree, however, with many of the other comments in Professor Krynine's discussion. Although in fact there are many minor points of disagreement, only the major ones will be discussed here.

First, it should perhaps be made clear that the purpose of the present paper was simply to indicate that compacted soils can possess pronounced thixotropic properties in spite of the suggestion by two previous investigators(1) that this would not be the case at the low water contents used in

a. Proc. Paper 1427, November, 1957, by H. B. Seed and C. K. Chan.
1. Associate Prof. of Civ. Eng., Univ. of Calif., Berkeley, Calif.
2. Asst. Research Engr., Inst. of Transportation & Traffic Eng., Univ. of California, Berkeley, Calif.

compacted soils and apparent confirmation of this suggestion by tests conducted by G. A. Leonards;(8) and also to indicate the influence of this thixotropic behavior in tests conducted over long periods of time. In spite of the evidence provided by these previous studies, Professor Krynine states initially that thixotropy "is to be expected" in compacted clays but later asserts that, "thixotropy is not an intrinsic property of a clay". It was with the hope of clarifying such apparently contradictory statements that the paper was presented.

Second, the writers do not consider that, with the consideration of electrical forces as a component of clay structure, "explanation of thixotropic processes becomes relatively simple". It would hardly seem adequate to simply attribute thixotropy to "structure building forces" without investigating further the mechanism of these forces. In fact it is believed that a comprehensive investigation of the cause of thixotropy would be a major contribution to fundamental knowledge of soil behavior.

Again the writers do not see that the stress-strain curves in Fig. 15 can be compared with the deformation-time curves in Fig. A and believe that Professor Krynine is confusing the time of testing with the time interval between compaction and testing when he states that the curve in Fig. 19A is a detail of one of the curves of Fig. 18.

There remains to be considered Professor Krynine's statements regarding the practical application of the results reported. Here again there seems to be some confusion of ideas since he states, "In a purely laboratory paper, such as the present one, it should be clearly shown what meaning the paper has for actual soil engineering, and how the phenomena observed in the cylindrical samples are reflected in actual earth masses. Without such a detailed explanation the paper may remain in the eyes of an average file and rank engineering reader as something abstract having no or almost no connection with the reality". Yet earlier he quotes Casagrande and Wilson, "The strengths of compacted soils . . . which are not fully saturated increase with time even when the water content is kept constant. From the standpoint of earth dam design these results have important practical applications", and adds that this opinion is shared by many field engineers.

Unlike Professor Krynine, the writers believe that readers of the paper would readily recognize the engineering significance of the results reported. Engineers are not likely to dismiss as "something abstract" a mechanism which provides an increase in strength for soils used for construction purposes. Many design tests on compacted clay samples, both for earth dam design and pavement design, are performed with only a short interval between compaction and testing. While such a procedure gives conservative results it is not necessarily economical.

Finally although the writers agree that care is required in translating the results of laboratory tests to field problems they must state their disagreement with Professor Krynine's opinion that "The topics discussed in a paper on soil mechanics should be connected with actual field experience or observations", and they disagree with the inference that research should have practical application if it is to be considered of value. If in fact this were the case, basic research in any field must be considered valueless. Such is certainly not the case. Rather it is often the fundamental investigations having no immediate practical application which provide the key to a fuller and broader understanding of phenomena having practical significance. An excellent example of this is the study of particle orientation in compacted soils

reported by T. W. Lambe. With the aid of this information many phenomena which previously were regarded as isolated pieces of information can now be fitted into an orderly pattern and used to predict the probable behavior of soils. It is certainly not the intent of the writers to suggest that the present paper falls under the category of basic research but there can be no doubt whatsoever concerning the value of fundamental investigations which, although they may have no immediate practical value and are not connected with actual field experiments or observations, ultimately provide the means for a better understanding of soil behavior.

The writers appreciate the discussions by E. S. Barber and D. P. Krynine.

LATERITE SOILS AND THEIR ENGINEERING CHARACTERISTICS[a]

Closure by K. S. Bawa

K. S. BAWA,[1] A. M. ASCE.—The author appreciates the interest shown in his paper. The discussions contributed have helped bring out not only the data concerning the engineering properties of laterite soils as found in different parts of the world but also experience gained with these soils during actual construction of different types of structures. Highway engineers in most tropical countries frequently come across these soils and hence the contributions of the Mr. Barber and Mr. Medina will be welcomed by many engineers. Much more data of similar nature is needed before any reliable correlation between the physical characteristics such as color, structure, grain size, etc., and the engineering properties such as plasticity, shrinkage (or swelling), shearing strength, compressibility, etc., can be attempted. The writer is in agreement with Mr. Medina's suggestion of setting up a body charged with collecting and disseminating engineering information (especially laboratory and field data) concerning laterite soils. The first task of such a body should be to standardize terms and definitions relating to laterite soils for engineering purposes. Due to world-wide contacts with the interested engineers, perhaps the most qualified agency which can handle such a task is the International Society of Soil Mechanics and Foundation Engineering, and aided if possible by one of the specialized agencies of the U. N., such as UNESCO. If there is sufficient interest and support, it might be possible to organize a subcommittee at the next (Fifth) conference of the Society to be held in Paris in the summer of 1961.

The author is thankful to Mr. Gizienski for contributing a thought-provoking discussion. The classification of laterite soils (as laterites and lateritic soils) was chosen arbitrarily as a starting point and like all other soil classifications has its limitations. In general, laterite soils are encountered as indurated crust formations, concretions intermixed with coarse and fine materials or clay-like soils depending on the nature of the parent rock and various factors of soil formation. The soil scientist's classification was selected because it actually rests on the degree of laterization (the delineation for laterites and lateritic soils being arbitrary) and it is the experience of engineers so far that certain properties of such soils can generally be anticipated once the degree of laterization is recognized. The author agrees with Mr. Gizienski that laterites can usually be identified by visual inspection. However, in identifying lateritic and non-lateritic soils, especially when both kinds are found in adjoining regions bordering the Tropics (see page 3 of original article, Proc. Paper No. 1428), considerable difficulty may be

a. Proc. Paper 1428, November, 1957, by K. S. Bawa.

1. Soils and Foundation Engr., Gannett Fleming Corddry & Carpenter, Inc. Consulting Engrs., Harrisburg, Pa.

experienced by the uninitiated engineer. The author is aware of the very general statements made on the basis of the limited data available at the time of writing the paper. As more and more pertinent data of the type reported by Florentin et al, (Reference 17 of Paper No. 1428) appears, the inadequacies of the earlier generalization can be corrected thus improving correlation between the physical properties and expected engineering behavior of laterite soils.

The corrugation in highway pavements in some tropical areas referred to by Mr. Gizienski can be ascribed to a variety of causes such as the presence of active clay mineral in addition to the lateritic nature of the soil. In fact, Mr. Wooltorton cites in his discussion to this paper the possibility of the presence of certain active clay minerals, such as montmorillonite or halloysite (considered to be an active form of kaolinite), in lateritic soils. Mr. Wooltorton also reports the crazing of pavements in soil areas (of tropical regions) where metahalloysite was found as clay mineral in the finer soil fraction.

GEOLOGIC INVESTIGATIONS OF DAM SITES BY THE SCS[a]

Closure by Gunnar M. Brune

GUNNAR M. BRUNE,[1] Aff. M. ASCE.—Thanks are extended to Mr. Trantina for his interest in the paper and for his discerning comments.

The Soil Conservation Service does not desire to compete with private industry in drilling and sampling investigations. Only in those states which have a heavy, full-time drilling work load and where it is clearly less expensive to the government, have Service drilling parties been used. In the great majority of states contract drilling by private industry is employed.

Mr. Trantina is quite correct in his statement that "If the assumed salaries of the geologists and drillers are lowered in order to raise the laborers' salaries to a realistic level, then obviously the salary-responsibility ratio is out of line and the man-in-charge is underpaid." At the time this paper was written, GS-7 and GS-5 geologists were used on the drilling parties. These men were not paid top-of-grade salaries and in many cases earned less than the drillers and junior drillers under their supervision.

This situation has recently been corrected. GS-9 and GS-7 geologists are now employed on the drilling parties. These men now receive premium (top-of-grade) salaries and in addition have recently received a 10 per cent raise in salary. The present salaries for a 7-man party are about as follows:

GS-9 Geologist in Charge	$ 6,875
GS-7 Assistant Geologist	5,869
Driller ($2.25 per hour)	4,680
Junior Driller ($2.00 per hour)	4,160
Three Laborers ($1.10 per hour)	6,330
Total annual salaries	$27,914

With this increase in salaries and wages, and taking into account other factors mentioned by Mr. Trantina, such as overhead costs, the average cost of drilling has increased to possibly $2.50 per foot. This is an average for all types of drilling and includes much more augering then denison sampling or rock coring. These latter types of drilling, of course, average much more per foot.

In regard to drill hole or trench spacing, the writer agrees that this depends upon the complexity of the geological pattern. However, there are other factors which must also be considered. Some of these are:

1. Previous construction experience in the area.

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a. Proc. Paper 1429, November, 1957, by Gunnar M. Brune.

1. Eng. Geologist, Engineering and Watershed Planning Unit, Soil Conservation Service, Fort Worth, Tex.

2. Classification of proposed structure in relation to hazards involved as a result of structural or functional failure.
3. Height and materials of proposed dam.
4. Amount of rock excavation involved.

On the subject of drive samplers, the writer did not intend to indicate that the Soil Conservation Service had closed the door on their use. These samplers are being used in some states and the Service is conducting studies for the purpose of developing guidelines for their proper use.

The writer wishes to thank Mr. Trantina for his careful analysis of this paper. He desires to stress again that the decision whether to use a Soil Conservation Service drilling party or a drilling contractor depends upon the circumstances in individual cases. The Soil Conservation Service will always be careful to choose the method which is most economical to the government, and if anything will lean toward the use of private enterprise for this work.

MODEL STUDY OF A DYNAMICALLY LATERALLY LOADED PILE[a]

Discussions by Lymon C. Reese and Hudson Matlock and John A. Focht, Jr.

LYMON C. REESE,[1] A. M. ASCE and HUDSON MATLOCK,[2] A. M. ASCE.—An important problem concerned with the design of offshore drilling structures is presented in this paper. As an example, in a recent design of a self-contained drilling platform it was estimated that the lateral load during a hurricane would be 5800 kips, while the vertical load on the structure during the drilling operation would be 11,300 kips. Little precedent exists for designing a structure (approximately 100 feet by 100 feet in plan dimensions) to resist such lateral loads. Any new information on this subject is welcome.

Mr. Gaul describes a series of tests in which cyclic loads were applied to a laboratory pile, and data are presented to show that no dynamic stress amplification occurred. It is reasonable to expect that the bending moments for a static load should agree with the bending moment determined for the first few cycles of a cyclic load of low frequency. However, the writers are strongly of the opinion that continued cyclic loading, say at the 91 pound load, would have caused a large increase in the magnitude of the bending moments.

In his discussion of the static test results Mr. Gaul states that "separation of the foundation at the mudline from one side of the pile was noted for all runs in which applied load was greater than about 60 pounds." This separation would have occurred for the cyclic loadings as well, and had there been water around the pile as for marine structures, deterioration of soil resistance in the vicinity of the pile would have occurred. The writers have observed this deterioration of soil resistance both in the laboratory and in the field. Mr. Gaul states that he was investigating a pile subjected to "dynamic" loading but his experiments and his discussion indicates that he was attempting to model a pile subject to cyclic loading from hurricanes. Readers should not make the unwarranted assumption that Mr. Gaul's experiments show that piles supporting offshore structures will develop no more bending moment under a large load applied cyclically by hurricanes than would be developed by a load of identical magnitude applied statically.

Mr. Gaul writes that "values of k in the relationship $E_s = kx$ have been estimated as low as 0.5 in very soft clay deposits," and that a value of k of 1 was chosen for use in designing the model. The value of k of 1 was also used later in the paper in performing analytical solutions. These points indicate that Mr. Gaul may believe that the parameter k is a soil property which is independent of load and pile restraint. An understanding of the nature of the parameter k can be obtained from Fig. 1. In Fig. 1-a is shown a pile which is

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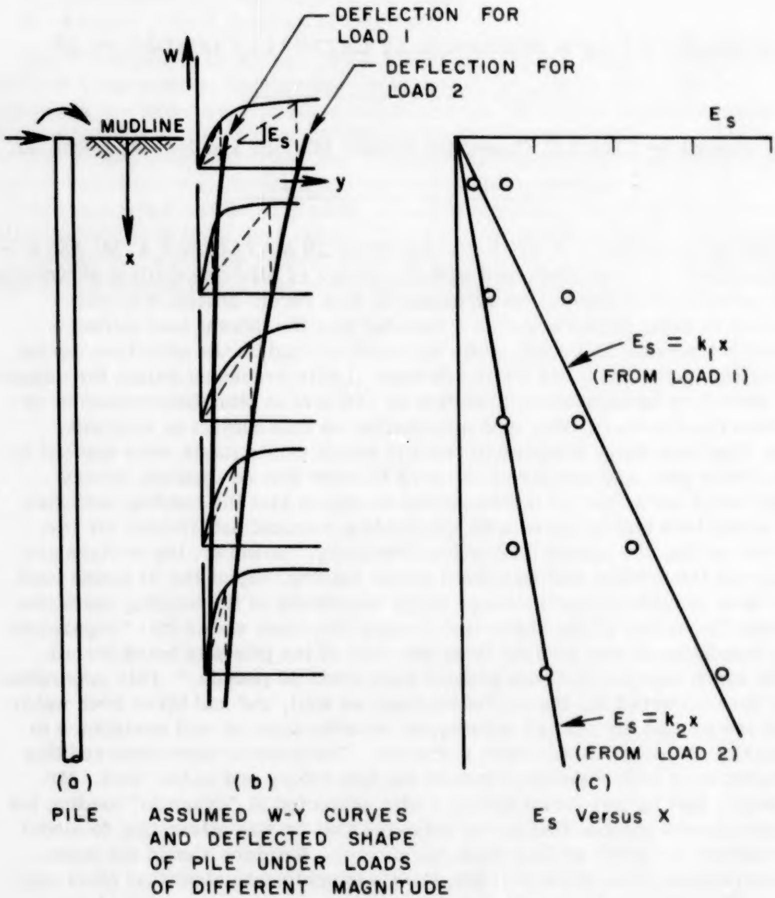
a. Proc. Paper 1535, February, 1958, by Roy D. Gaul.

1. Assoc. Prof. of Civ. Eng., Univ. of Texas.
2. Assoc. Prof. of Civ. Eng., Univ. of Texas, Austin, Tex.



DETERMINATION OF SOIL MODULUS
FOR A LATERALLY LOADED PILE

FIGURE 1

subjected to a thrust and a moment at the mudline. In Fig. 1-b is shown an assumed set of w versus y relationships for the soil at several points along the pile, where w is the soil resistance against the pile and y is the pile deflection. The shape of these w-y curves is consistent with that proposed by McClelland and Focht.(1) Superposed on these w-y curves are assumed deflected shapes for the pile for two loadings. The value of E_s for each loading and for each w-y curve can be obtained graphically as shown in Fig. 1-b. These possible values of E_s are shown plotted versus depth in Fig. 1-c. Fig. 1-c shows that the parameter k will be relatively large for loads of low magnitude and will be reduced as the load is increased.

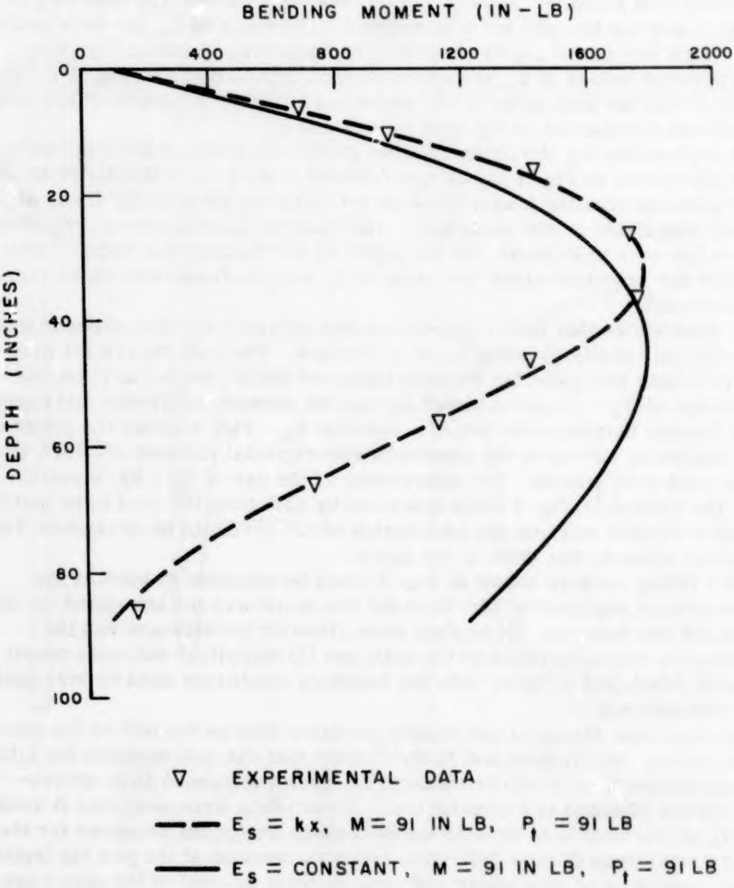
The explanation for the determination of the soil modulus for a laterally loaded pile shown in Fig. 1 is not complete but it does show that there is little justification for treating k as a constant for different piles or for loads of different magnitude on the same pile. The value of E_s will remain relatively constant for very light loads, but the paper by McClelland and Focht(1) indicates that for practical cases the value of E_s will decrease with an increase in applied load.

Mr. Gaul states that better agreement was obtained between experimental and analytical results by using E_s as a constant. The writers are not prepared to debate this point for reasons indicated below, but for at least one load the use of $E_s = kx$ gives better agreement between analytical and experimental results than does the use of a constant E_s. Fig. 2 shows the results of fitting analytical curves to the maximum experimental moment obtained with a static load of 91 pounds. The superiority of the use of $E_s = kx$ is readily seen. The curves in Fig. 2 were computed by assuming the load to be applied one inch above the mudline, an assumption which seems to be consistent with statements made by Mr. Gaul in his paper.

Curve fitting such as shown in Fig. 2 could be extended to each of the moment curves reported by Mr. Gaul but this work was not attempted by the writers for two reasons: (1) no data were given on the strength and the stress-strain characteristics of the soil, and (2) analytical solutions would need to be developed to agree with the boundary conditions used by Mr. Gaul in his experiments.

It is hoped that Mr. Gaul can supply complete data on the soil in his closing discussion. McClelland and Focht(1) state that the soil modulus for laterally loaded piles in clay can be obtained in certain instances from stress-strain curves obtained in a triaxial test. If such data were available it would probably be worthwhile to develop the necessary analytical solutions for the boundary conditions of zero deflection and zero moment at the pile tip (rather than the conditions of zero shear and zero moment as used in the above curve fitting) and to make analytical studies in order to obtain additional insight into the behavior of the soil surrounding a laterally loaded pile. Without these essential soil data, analytical studies of the sort shown in Fig. 2 would serve little purpose.

The author states that no dynamic load factor need be used in obtaining the bending moments in a pile subjected to dynamic loads of the frequencies used in his experiments; however, the reader should be aware that this conclusion does not apply to the usual case where a number of piles support a structure. Newmark (2) has discussed the situation where pile-supported structures are subjected to dynamic loads and has shown that the dynamic response of a structure may differ markedly from the static response.



ANALYTICAL CURVES FITTED TO EXPERIMENTAL DATA FOR STATIC LOAD OF 91 POUNDS

FIGURE 2

REFERENCES

1. McClelland, Bramlette, and John A. Focht, Jr., "Soil Modulus of Laterally Loaded Piles," Proceedings, ASCE, Vol. 82, Paper 1081, October 1956.

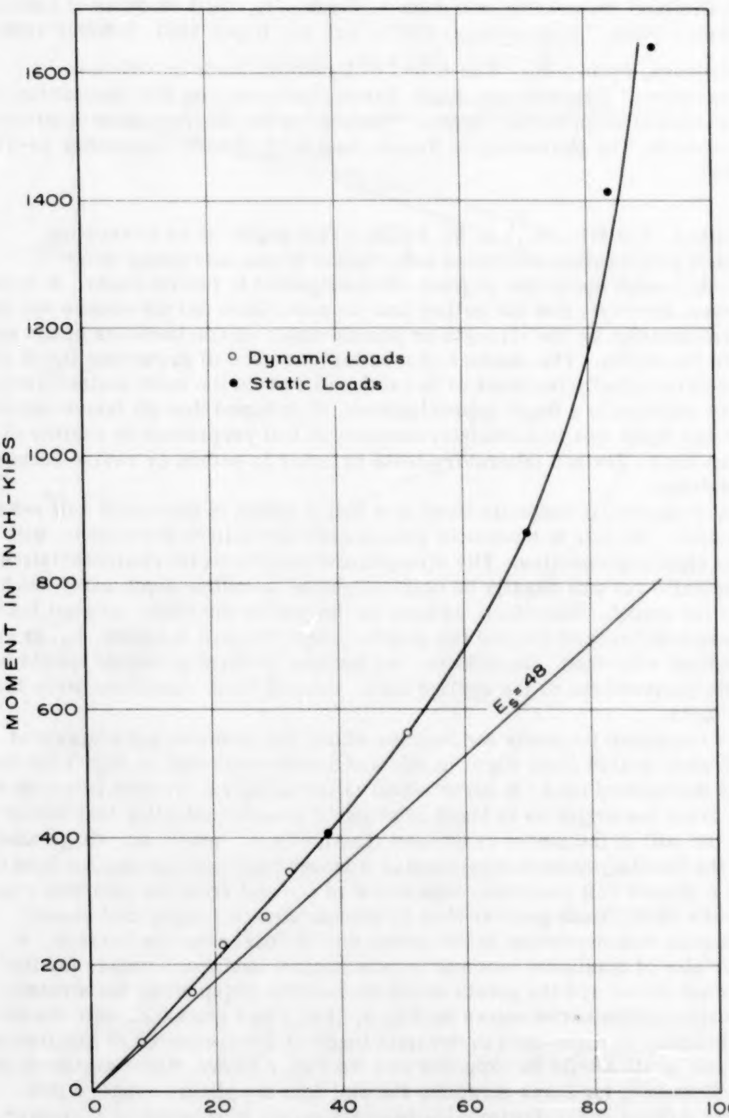
2. Newmark, Nathan M., "The Effect of Dynamic Loads on Offshore Structures," Proceedings, Eigth Texas Conference on Soil Mechanics and Foundation Engineering, Special Publication No. 29, Bureau of Engineering Research, The University of Texas, Austin 12, Texas, September 14-15, 1956.

JOHN A. FOCHT, JR.,[1] A. M. ASCE.—This paper on an interesting research project adds additional information to our increasing store of knowledge about the action of piles when subjected to lateral loads. It is unfortunate, however, that the author and his associates did not secure any specific information on the strength or stress-strain characteristics of the soil used in the model. The absence of a suitable method of predicting the in situ stress-strain characteristics of the soil will reduce the most mathematically precise solution to a mere approximation. It is hoped that all future experiments and tests will include determination of soil properties by routine or even specially devised laboratory tests in order to extend or revise existing correlations.

The commercial bentonite used as a soil medium in the model will set up in a manner similar to household gelatin and, for limited deflections, will exhibit elastic properties. The strength and stress-strain characteristics of the bentonite gel will usually be uniform within a shallow depth range such as that of the model. Therefore, as long as the soil in the model was not loaded to produce deflections beyond the elastic range, the soil modulus, E_s, should be constant with depth. In addition, the bending moment produced should be directly proportional to the applied load. Both of these conditions were found in the tests.

The maximum moments for both the static and dynamic applications of Test 2 were scaled from Figs. 7a and 9 and were replotted on Fig. 1 (writer) versus the applied load. A curve which is essentially a straight line may be drawn from the origin up to loads of about 50 pounds indicating that within this range the soil in the model responded elastically, or nearly so. At greater loads the bending moment increases at a faster rate than the applied load indicating a plastic soil reaction. Separation of the soil from the pile was reported for the static loads greater than 60 pounds also indicating that plastic deformation was occurring in the model soil at least near the surface. A similar plot of maximum moment versus applied load can be made for the other load series and the points would be found to group along the straight-line portion of the curve shown on Fig. 1. For loads producing only elastic deformations, no difference in dynamic loads of low frequency of oscillation and static loads should be expected and, as Fig. 1 shows, there was no difference. However, for loads straining the soil into the plastic range, a progressive failure under dynamic loads might result in substantially greater moments than were produced by the single application of static loads. In the tests reported, the dynamic loads were all within what might be termed the

1. Chf. Design Engr., McClelland Engineers, Inc., Houston, Tex.



MOMENT vs LOAD
TEST 2

Fig. 1

elastic range; therefore, additional tests are necessary to determine if different bending moments would result from static and dynamic loads which stress the soil into its plastic range.

No data were included in the paper as to the magnitude of the deflections which occurred at the soil surface. It would be informative if such data could be supplied, particularly to indicate the deflections at which separation of soil from the pile began.

A soil modulus which increases with depth will be found if the foundation medium increases in strength and stiffness with depth similar to a full-scale sand foundation or a deep normally consolidated clay deposit. A modulus increasing with depth may also be developed if the soil near the surface is stressed beyond the elastic range into its plastic range. Because the soil in the model was essentially a uniform elastic medium and because the dynamic loads did not strain the soil beyond the elastic range, moment distributions computed assuming a constant soil modulus showed reasonable agreement with the test data. The maximum moment computed for E_s = 48 lb per in^2 is plotted versus applied load on Fig. 1. It is suggested, therefore, that the last conclusion on page 1535-31 be revised to emphasize that reasonable agreement was obtained between the theoretically computed moments for a constant soil modulus and the moments developed in this test for both static and dynamic loads. For conditions other than those of these tests, the most reasonable fit of test and theoretical moment distributions might be obtained using E_s equal kx or some other variation of E_s with depth.

A METHOD TO DESCRIBE SOIL TEMPERATURE VARIATION[a]

Corrections by D. C. Pearce[1]

CORRECTIONS.—On page 1. The formula $\sqrt{\frac{p}{\pi\alpha}}$ should read $\sqrt{\frac{\pi}{\alpha p}}$.

a. Proc. Paper 1537, February, 1958, by E. B. Penrod, W. W. Walton and D. V. Terrell.

1. Building Services Section, National Research Council of Canada, Ottawa, Canada.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: GROUTING WITH CLAY-CEMENT GROUTS[a]

Discussion by Bruce E. Clark

BRUCE E. CLARK,[1] Aff. M. ASCE.—The only important fault the writer finds in this paper is its brevity. He would particularly have liked to see more discussion of the reasons for and against using clay in grout for various types of jobs. In addition to the above, the writer would like to comment on a few specific statements of the author's, in the paragraphs that follow:

Page 4, Penetration:—The author states that the groutability ratio (D_{15} formation/D_{85} grout) should be more than 25. This is a rather safe value. The minimum groutability ratios found by the Waterways Experiment Station for various types of cements(1) varied from 11 to 24, probably depending largely on the shapes of the gradation curves of the grout and the formation. Also, Kravetz mentions a groutability ratio of 5 to 20 for clay on page 11 of his paper (ASCE Proc. Paper 1546). Groutability of fine cracks, on the other hand, appears to depend on the maximum particle size in the grout. According to recent studies,(2) the ratio of crack width to maximum grout particle size varies from 1.7 minimum to 3.0 or more.

Page 4, Mix Design, 1st par.:—It is stated that "The addition of fly-ash or silt will also help to reduce segregation." This is quite true, but so will more cement, and to nearly the same degree. Therefore, unless the net cost of fly-ash or silt is less than that of cement, there is little advantage in their use. The cause of segregation is lack of plasticity of the grout, usually caused by too much water, or by too little of the plasticity-giving materials, such as cement, fly-ash, diatomite, silt, or clay. The water content of a sand-cement grout is quite critical.

Page 4, Mix Design, par. 4-5:—The author here discusses the pros and cons of choosing between thin or thick neat cement grouts. To the writer, this choice appears to be largely a field problem. In grouting fine openings, it is certainly better to inject thin grout than none, so the thing to do is to put in grout as thick as the openings will accept. Much of the argument over thin versus thick grout appears to be the result of disregarding the differences between different jobs, or between job and laboratory. In some cases, the use of thin grouts will certainly result in a poor job. However, there have been many instances where job conditions evidently favored separation of the excess water from high W/C grout, resulting in openings tightly filled with hard grout. An example of this is Simonds' reference to Grand Coulee on page 5, 4th par., and Fig. 3 (ASCE paper 1544).

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a. Proc. Paper 1545, February, 1958, by Stanley J. Johnson.
1. Dist. Geologist, Nashville Dist., U. S. Army Corps of Engrs., Nashville, Tenn.

Page 5, Strength:—The author says "The compressive strength of a grout is generally determined on the basis of the grout as mixed in the laboratory or in the field, which neglects the loss of excess water and the resultant increase in strength which generally occurs during injection." The assumption apparently implicit in this statement that grout in the ground loses water, whereas grout in a test cylinder does not, seems unwarranted. Grout in a test cylinder will lose water, unless it is an exceptionally stable mix which would probably lose little water in the ground. It appears to the writer that grout in the ground is apt to lose either more or less water than the same grout in a test cylinder, depending chiefly on the size and arrangement of the openings underground, and to a lesser extent on grouting pressures used.

Page 6, Grouting Pressures:—The statements by the author on this subject are true, but not very helpful. The writer believes that arbitrary pressure limits are quite often too low and that field experiments on pressure are frequently desirable, unless structural damage is likely. In the latter case, conservative pressures are usually indicated.

Page 11, Fig. 4:—In this figure, the envelopes shown by the author for sand or clay versus water should not intersect the origin as shown, since the one sack of cement specified also requires water—at least 0.5 or 0.6 cubic foot.

REFERENCES

1. "Grouting of Foundation Sands & Gravels", Tech. Memo. 3-408, Waterways Experiment Station, Vicksburg, Miss., June 1955.

2. "Pressure Grouting Fine Fissures", Tech. Report 6-437, Waterways Experiment Station, Vicksburg, Miss., October 1956.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: THE USE OF CLAY IN PRESSURE GROUTING[a]

Discussion by Bruce E. Clark

BRUCE E. CLARK,[1] Aff. M. ASCE.—The author has presented a very interesting paper, and the fact that so little has been published in the United States on the theoretical side of clay grouting makes this paper especially valuable. However, the writer would have liked to see more information to help in deciding whether clay should be used for a particular grouting job or not. In the hope of eliciting such information the following comments are offered, setting forth some of the objections to the use of clay grout:

Clay grout has been used for the following reasons:

1. To penetrate openings too fine for portland cement;
2. To stabilize suspension grouts, particularly those containing sand;
3. To make a cheap grout where large openings must be filled (reasons 2 and 3 often go together); and
4. To make a grout more compatible with earth foundation or embankment than is hard, rigid portland cement.

Unfortunately, the use of clay in grout is open to several objections. Thus, to penetrate openings too fine for portland cement, clay grout must be diluted to the point that the percentage of solids is very low (particularly for high liquid limit clays like bentonite). The writer has never seen evidence that enough excess water can be driven off from clay grout during grouting to make a stiff clay, yet unless it can, the grout could hardly be expected to be very permanent, and the addition of even a little portland cement will practically limit the penetration of grout to the openings large enough for the cement.

Clay is quite effective in stabilizing suspension grouts, although it is certainly not necessary to add clay to a sand-cement grout for stability. For example, the writer recently field-designed a sand-cement grout with the following mix design:

Sand: 2.2 CF; Cement 1 CF (bag); Water: 0.8 CF

The sand was a commercial crushed limestone mortar sand with the following gradation:

Screen	% Pass	Screen	% Pass
# 4	100	# 50	19.15
# 8	95.87	#100	8.22
# 16	68.47	#200	2.74
# 30	38.33		

a. Proc. Paper 1546, February, 1958, by Glebe A. Kravetz.

1. Dist. Geologist, Nashville Dist., U. S. Army Corps of Engrs., Nashville, Tenn.

This was mixed in a Colcrete mixer at 1500 rpm. The grout was plastic, but easily pumped, by the centrifugal pump action of the mixer alone. A 28-day test cylinder of this grout mix broke at 4368 psi and the mix showed no measurable bleeding shrinkage. Although the high strength of this grout was not particularly needed, the plasticity and lack of bleeding shrinkage were, and it is hard to see how a clay grout could have been more satisfactory in respect to these properties.

The chief advantage of clay grout therefore appears to be cheapness. Clay itself is not particularly cheap, by the time it is processed and mixed in grout. Except for large jobs, clay is best bought dried, pulverized, and sacked. Processed clay is readily obtained, as it is widely used for drilling mud and for the manufacture of bricks, tile, etc. The current price of lean clays in Nashville, Tenn., is $1.25/100 lb. sack, and of bentonite is $2.40/100 lb. sack. This sounds about as expensive as cement, but one sack of lean clay will carry as much sand as 4 or 5 sacks of cement, and one sack of bentonite will carry more sand than 30 sacks of cement. As a result, clay reduces the overall cost of grouting materials greatly. Thus, the following costs of materials per cubic foot of grout were calculated for workable mix designs:

Mix	Cost per Cubic Foot
1 cement, 2 clay, 17 silty sand	$0.38
1 cement, 2 sand	0.75
Neat cement	1.30

The cement-clay-sand grout gave a product nearly like soil, whereas the cement-sand and neat cement grouts set up almost like rock.

The fourth advantage of clay grouts—that they are more compatible with earth—is more or less true, though the advantage of grout being as weak as the surrounding soil is rather debatable. Furthermore, all clay grouts, unless a great deal of water can be squeezed out of them, have very high void ratios. If they contain cement, the strength can be as high or higher than the surrounding soil, but the permanency of such a light, porous material seems a bit dubious. Certainly, if the strength of the cement lattice were exceeded, it would show extremely high consolidation.

In addition to the above general discussion, the writer would like to comment on specific statements in the paper in the paragraphs following.

Page 3, Introduction, par. 3:—In this paragraph, the statement is made that "The use of clay . . . improves the quality of a basic cement . . . mix . . ." This, the writer doubts. Some qualities are improved, yes. But strength, impermeability, and resistance to leaching are always adversely affected by the presence of clay, which fact must be considered in planning clay grout mixtures.

Page 6, Grout Mixtures, par. 1:—The author makes the statement that "Coarse grouts can be defined as mechanical suspensions, meaning that only mechanical action can maintain them in that state." This appears rather confusing, since most practical coarse grout mixes contain a plasticizing agent such as cement or clay. Such grouts can be designed to stand for appreciable periods without segregation or undue stiffening. The only grout the writer can think of that would fit the above definition would be a high W/C non-plastic neat cement grout (W/C above 0.9 CF/bag).

Page 9, par. top of page:—Apparently the author is not aware that the writer measured "yield value" and "thixotropy" of cement grout several years ago.(1)

Page 9, Viscosity, Rigidity and Thixotropy Test, last par.:—The author makes the statement that it is better not to measure the density of a grout with a hydrometer. This is correct for plastic grouts. However, the density of non-plastic grouts (such as neat cement suspensions with a W/C above approximately 0.9 cubic foot/bag) can be measured accurately with a hydrometer, provided it is done quickly before sedimentation has time to occur.

Page 11, Clay Grout, par. 3:—The statement that "Sometimes sand foundations cannot be grouted by cement mixtures, not because of the particle to void ratio requirement, but because of the low stability of the cement mixtures." does not agree with the findings of the Waterways Experiment Station.(2) In their tests, the groutability ratio "N", which was reported for various types of cement, varied from 11 to 24.

Page 18, Cement Grouting, 2nd par.:—As stated, high speed mixing improves cement grout—and clay grout also—(by reducing the yield and the plastic viscosity) but the cause does not appear to be that it activates the hydration of the cement, but rather thixotropic behavior of the cement-gel colloids, for high speed mixing has no effect on setting time.

Page 21, top par.:—Contrary to the author's statement here, extensive tests have shown that properly designed cement-sand grouts can be pumped without segregation.(3) The value of high speed mixing (commercially called colloidal grout) is recognized, as is the value of fly-ash, diatomite, silt, clay, etc., but none of these things is a substitute for good mix design.

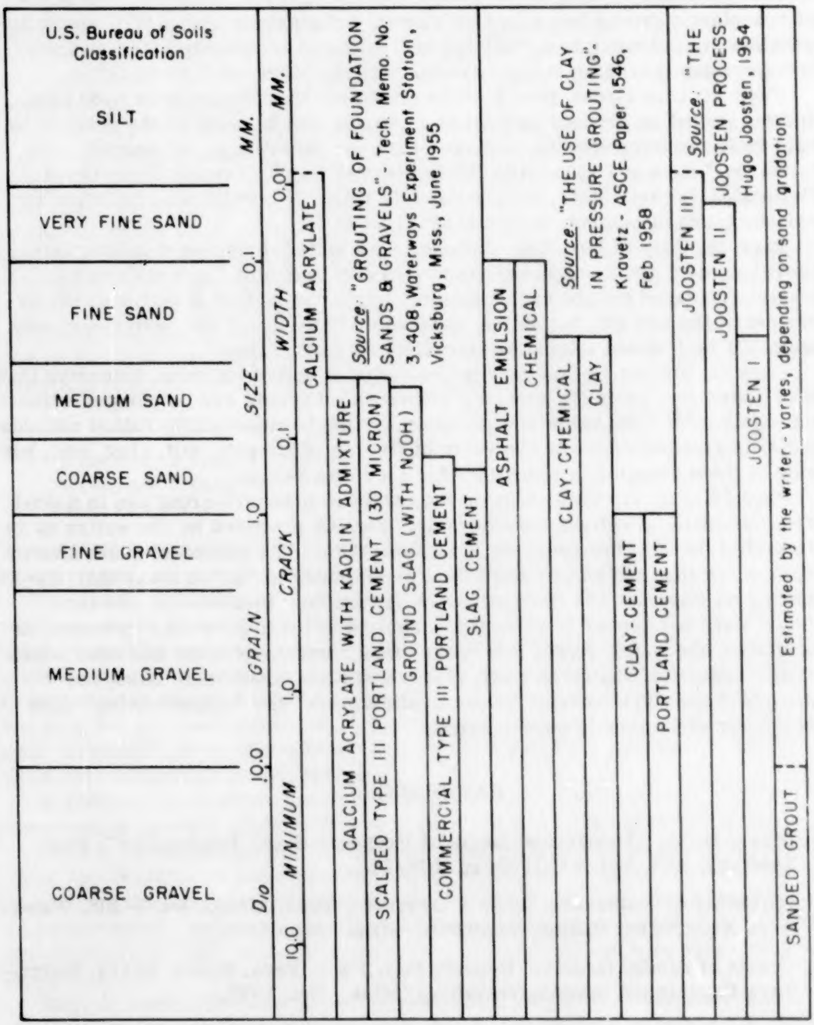
Page 25, Fig. 1:—This chart on the extent of pressure grout use is a good idea, but could be refined considerably. Fig. 1A prepared by the writer is an attempt to do this, incorporating additional data. The minimum grain size or crack width that can be grouted with a given material varies somewhat, depending on material and field technique variations. Incidentally, the term "fault" does not appear to be correct, applied to large groutable openings, as the author has done. Faults are shear displacements of rock, and fault zones usually consist of shattered rock, clay, or similar materials. Also, the author's distinction between "impermeabilization" and "consolidation" does not appear necessary in such a chart.

REFERENCES

1. Clark, B. E., "Theoretical Basis of Pressure Grout Penetration", Proceedings, ACI, Vol. 52, 1955, pp. 215-24.

2. "Grouting of Foundation Sands & Gravels", Tech. Memo. No. 3-408, Waterways Experiment Station, Vicksburg, Miss., June 1955.

3. "Tests of Sanded Grouts," Reports Nos. 1 & 2, Tech. Memo. 6-419, Waterways Experiment Station, Vicksburg, Miss., Oct. 1955.



PRESSURE GROUT SELECTION CHART

Fig. 1A

CEMENT AND CLAY GROUTING OF FOUNDATIONS: THE USE OF ADMIXTURES IN CEMENT GROUTS[a]

Discussion by Bruce E. Clark

BRUCE E. CLARK,[1] Aff. M. ASCE.—The authors have presented an excellent review of the subject of admixtures in grout, and their findings are in substantial agreement with the writer's observations. However, a few comments are in order about their use of various rheological terms. The writer found several parts of the paper almost incomprehensible until he finally realized that rheological terms were being used in a different sense from that to which he is accustomed. The best source of information on rheology (the science of flow, particularly of viscous liquids and plastics) known to the writer is Green.(1) The following discussion is based on the usage of his book.

Plastic:—The authors say on page 11, "Plastic grouts are those which will not flow initially of their own weight, but which require an initial force (called the 'yield value') to start flow." This definition describes a material like putty. Only the stiffest grouts fit such a definition, grouts almost too thick to pump. Actually, gravity has nothing to do with the definition of a plastic (also called Bingham body or non-Newtonian). A liquid shears in response to an infinitesmal force. A plastic does not shear until the yield value is exceeded. All stable mortar grouts are plastic, as are neat cement grouts with a W/C less than about 0.9 cubic foot/bag. The authors' paper is concerned primarily with plastic grouts, according to this definition, and contrary to their statement on page 3, Grout Mixtures, par. 3.

Consistency and Fluidity:—According to Green, consistency and fluidity are reciprocals. The writer prefers the term "mobility" for plastics and "fluidity" for liquids, or their reciprocals, "plastic viscosity" and "viscosity". Plastics have a complex flow characteristic curve like Kravetz' (ASCE paper 1546) Fig. 2, and no really satisfactory way of expressing it except in a graph has been found. The two tests the authors describe as consistency and fluidity (the consistency meter and flow cone, respectively) are useful tests for plastic grouts, but merely measure functions of one point on the flow characteristic curve. It is therefore incorrect to think of these tests as measures of different properties of the grout, as the authors apparently do, when the tests are actually different, partial measures of the same property: plastic viscosity. Furthermore, the authors apparently picture the flow of plastic grout through a pipe as consisting of a solid body of grout moving with wall friction but no hydraulic shear (as is practically the case with pumped concrete). Actually,

a. Proc. Paper 1547, February, 1958, by Alexander Klein and Milos Polivka.

1. Dist. Geologist, Nashville Dist., U. S. Army Corps of Engrs., Nashville, Tenn.

grout must be very stiff and be pumped very slowly to flow this way. In a more typical mode of plastic or plug flow, there is a non-shearing core and an outer zone where hydraulic shear occurs. As the speed of pumping increases, the size of the core decreases. The flow of plastics is covered thoroughly by Green in chapter 2 of his book.

The above misuse (according to the writer's thinking) of terms may not seem important. It certainly has little effect on the value of the paper. Furthermore, a similar loose use of words has been prevalent in other industrial applications of rheology. However, the writer feels that the scientific rheologists have much to offer the field of grouting, and that a common language with precise definitions of terms is essential to progress.

Flow Cone Test:—The flow cone test has one serious fault not mentioned by the authors: Its results depend as much on the specific gravity as on the consistency of the grout. The data showing a correlation between flow cone and consistency meter tests (Fig. 5) do not agree with data obtained by the Waterways Experiment Station,(2) which showed little correlation. The writer's experience with a flow cone has been that it is not very sensitive, but is invaluable for field mix design, as it helps to detect a tendency towards segregation.

REFERENCES

1. Green, Henry: "Industrial Rheology and Rheological Structures", John Wiley & Sons, Inc., N. Y. C., 1949.
2. "Pressure Grouting Fine Fissures", Tech. Report No. 6-437, Waterways Experiment Station, Vicksburg, Miss., October 1956.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: SUGGESTED SPECIFICATIONS FOR PRESSURE GROUTING[a]

Discussions by Bruce E. Clark and L. A. Schmidt, Jr.

BRUCE E. CLARK,[1] Aff. M. ASCE.—The use of a guide is essential for efficient preparation of a grouting specification, even though each job has different requirements. This excellent guide specification is one of the most comprehensive the writer has seen, and although it resembles that of the Corps of Engineers fairly closely, the writer certainly plans to use it for reference. There are a few minor comments the writer would like to make.

Packer Grouting:—Although mentioned in definitions, this specification makes no provision for use of a packer. It is suggested that two schedule items be provided for connection to foundation grout holes, one for stage grouting, one for packer grouting; or some other arrangement might be made which would permit use of packers as directed. Experience has shown that there are places, not always predicted in advance, where a packer is very advantageous.

Page 4, items 7 and 8:—Apparently these items (drilling AX holes in rock) are intended for grout hole drilling. If so, they are probably better called "Drilling grout holes in rock, _X size." In either case, it is hard to see the justification for holes larger than EX (1-1/2 inch) except for good reason, for EX grout holes have proven amply large for nearly all purposes on many major jobs.

Page 9, par. 5h:—As stated, 1-1/2-inch grout lines are usually desirable, but at Folsom project, the writer found it perfectly possible to force neat grout through 1/2-inch pipe for circuit grouting (as shown in Burwell's ASCE paper 1551, Fig. 3). The Waterways Experiment Station used 200 feet of 3/4-inch I.D. hose in its mortar grout experiments,(1) and found that a well designed grout mix could stand in it for 15 minutes, after which pumping could be easily resumed.

Page 10, par. 9:—Since screening of cement is rarely done, it might be pointed out here that wet screening of grout (easily done with a vibratory screen) is far cheaper than dry screening.

Page 10, par. 11:—Sand grading specifications should be based on the best sand locally available. A well graded sand scalped on the # 16 sieve seems to be best, but comparative tests are desirable. The sand which permits the lowest C/S and W/C ratios in a stable mix is best, provided its cost is not out of line.

Page 14, par. 20:—AX holes yield a 1-3/16-inch core, not 1-15/16-inch!

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a. Proc. Paper 1548, February, 1958, by Judson P. Elston.

1. Dist. Geologist, Nashville Dist., U. S. Army Corps of Engrs., Nashville, Tenn.

Page 28, par. 43:—Screening grout made of ordinary cement (type I or, preferably, type III) through a 200 mesh vibrating screen should be better and cheaper than using rescreened cement, as specified here. Better yet, it is possible for some cement mills to prepare a special cement with no more than 4% over 30 micron size. One manufacturer estimated a premium of only 50¢/bbl. over standard type III for this cement, if ordered in quantity.(2)

REFERENCES

1. "Tests on Sanded Grout, Report No. 1", Tech. Memo. No. 6-419, Waterways Experiment Station, Vicksburg, Miss., October 1955.

2. "Grouting of Foundation Sands and Gravels", Tech. Memo. No. 3-408, Waterways Experiment Station, Vicksburg, Miss., June 1955.

L. A. SCHMIDT, Jr.,[1] M. ASCE.—Specifications and contract documents as described by Mr. Elston can be made entirely too cumbersome. Successful grouting jobs, at best, require somewhat trial-and-error methods and improvisation is often the key to the solution of a specific grouting problem. Unfortunately most clients require drilling and grouting to be done on a contract unit price or lump sum basis. Much argument and substantial proof can be advanced that core drilling and grouting are engineering problems that can best be handled by foundation engineers working with contractors on a cost plus or upset hourly rate basis. The requirements for well-treated foundations, unlike man-made structures, cannot be calculated with the accuracy with which one can figure the cost of constructing a dam or a building. A structure above ground belongs to the contractor until it is accepted by the owner. At no time does the foundation belong to the contractor. From the time work is started until it is completed, the foundation has always belonged to the owner. Unless the owner is willing to accept the risks inherent in making such a foundation adequate for its purpose, he is apt to find himself with an inferior job if the contractor is losing money or with a job that has cost entirely too much because of the contingency factor for which the contractor necessarily has allowed. In addition to these practical considerations are the factors of unnecessary costs of engineering to prepare specifications, drawings and contract documents of such scope as to cover every foreseeable contingency. For this portion of construction work, a great deal can be said for the force account system of work as practiced by TVA where foundation treatment methods and operations are developed in the field at any particular job for a specific purpose.

1. Pres., Schmidt Eng. Co., Inc., Chattanooga, Tenn.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: PRESSURE GROUTING WITH PACKERS[a]

Discussion by Bruce E. Clark

BRUCE E. CLARK,[1] Aff. M. ASCE.—The author's presentation of the case for (and to some extent against) packer grouting is remarkably fair, considering that he starts out by stating that packer grouting is a great improvement over stage grouting. However, it seems to the writer that the long standing controversy over packer versus stage grouting ought to be settled by compromise. Every grouting job presents its own problems and the best place to solve them is on the job. The grouting work the writer has been associated with could largely be divided into four categories: grouting best done by the stage method; grouting best done by the packer method; grouting best done by a combination of the two methods; and grouting which could be done about equally well by either method. For example, packer grouting is rather impractical in certain highly jointed formations; the packers are too hard to seat, and when seated, are too frequently bypassed by the grout. Further, it is very difficult to drill deep holes in many formations without cementing because of caving. All such highly jointed and/or caving formations are best treated by stage grouting. On the other hand, some formations will not hold the successively higher pressures imposed by stage grouting and must be packer grouted to avoid repeated grout breakouts. It is not always possible to determine in advance which method should be used on a particular job, and it may often be desirable to use both methods on the same job. Therefore, the writer suggests that grouting specifications provide for both stage and packer grouting, as directed, so that the decision on how to grout the job can be made on the job, on the basis of field experience.

Incidentally, the cup type packer shown by Burwell (ASCE paper 1551, Fig. 6) is an improvement over the one shown by the author in that being constructed of aluminum, it can be drilled out in the event it gets grouted in the hole accidentally.

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a. Proc. Paper 1549, February, 1958, by Fred H. Lippold.

1. Dist. Geologist, Nashville Dist., U. S. Army Corps of Engrs., Nashville, Tenn.

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CEMENT AND CLAY GROUTING OF FOUNDATIONS: FRENCH GROUTING PRACTICE[a]

Discussion by Bruce E. Clark

BRUCE E. CLARK,[1] Aff. M. ASCE.—This interesting paper confirms the impression that grouting of pervious soils has been much more widely practiced in Europe than it has in the United States. Without having had experience in both countries, one can only speculate on the reasons for this situation. Cheaper relative costs of alternatives such as well-point systems, trench and backfilling, etc., may be part of the reason. But, it must be admitted that ignorance, in the United States, of methods of grouting pervious soils is bound to be an important reason why we do so little grouting of them.

The cost of grouting alluvium is evidently very high. But, the impression given by the tabulation of the surface of cutoff—300 m^2 = 3000 sq. ft. at Genissiat Dam, is of a cutoff (for example) 100 feet long by 30 feet deep, which must surely be incorrect. The actual dimensions of the cutoff would be appreciated. Complete similar data for the other projects described would also be of interest.

The statement on page 3, next to last paragraph, to the effect that high-speed mixers change the granular composition of cement cannot be let go unchallenged. Anyone who examines cement under the microscope will be struck by its resemblance to sand. The grains are not only angular, they are hard. Rivers, no matter how violent their turbulence, are quite unable to reduce the size of sand grains, because of the cushioning action of water. It is hard to see how any grout mixer could break down cement grains mechanically, for they are already much finer than sand. The writer has investigated high speed mixing and believes it is quite beneficial to grout. But the effects of high speed mixing appear to result from a thixotropic alteration in the colloids in the grout (such as cement-gel and clay).

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a. Proc. Paper 1550, February, 1958, by Armand Mayer.

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CEMENT AND CLAY GROUTING OF FOUNDATIONS: EXPERIENCE OF TVA WITH CLAY-CEMENT AND RELATED GROUTS[a]

Discussion by L. A. Schmidt, Jr.

L. A. SCHMIDT, Jr.,[1] M. ASCE.—The symposium on clay cement grouting of foundations has made a real contribution to foundation-treatment methods. Often this mixture of materials can be used to effect desirable results at economical costs. The use of this mixture is not, however, a universal panacea for grouting all types of foundations. Messrs. Leonard and Grant cite limitations in its use and wisely prescribe caution for its application.

Another point made by these authors that cannot be emphasized enough is that the use of low pressures is important in grouting areas where the foundation characteristics are such as to warrant the use of clay cement grout. Obviously the application for such grout is in areas where there is no running water and where cavitation is so prevalent that the use of only cement grout becomes economically burdensome. By its very nature such a foundation will absorb enormous amounts of grout, and if pressures are kept high, the grout will often travel so far afield that much of it has no beneficial effect on the immediate requirement. The foundation under the log pond dike at Bowaters' Southern Paper Corporation plant at Calhoun, Tennessee, for example, was one where the rock was so open that not enough places were found where packers could be set. The foundation was grouted with clay cement grout through open end pipes under atmospheric pressure. Excellent results were obtained.

Experience has demonstrated that the use of the type packer described by Messrs. Leonard and Grant is superior to most other types of packers. It is a simple device that can be set accurately without time-consuming effort. The entire assembly may be machined at one time from stock materials without regard for the set screw arrangement in the bottom collar as shown on the illustrated figure. Present day practices in this area are to stretch the rubber packer over the bottom shoulder. This can be done conveniently and eliminates the need for the collar to be a separate piece.

It would be extremely valuable if the amount of grout taken per hole and per foot of hole in the various stages of grouting could be given for the projects described by Messrs. Leonard and Grant. If this data is available, it is hoped that the authors will be able to present tabulations of it in their closing discussion. Such a record is indicative of the effectiveness of the grouting provided the grout takes become less as the spacing of the holes is decreased. From this record it is also possible to determine reasonably well

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a. Proc. Paper 1552, February, 1958, by George K. Leonard and Leland F. Grant.

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what the spacing for primary holes should actually be in the type of formation that is being grouted.

THE STRUCTURE OF COMPACTED CLAY[a]

Discussions by B. P. Warkentin and R. N. Y. Yong and G. A. Leonards

B. P. WARKENTIN,[1] and R. N. Y. YONG,[2] J. M. ASCE.—Dr. Lambe has presented a very stimulating and informative discussion on the structure of compacted clay, based upon recent studies in his laboratories and in others, of both the mechanical and physico-chemical behaviour of clays. It is particularly gratifying to everyone interested in these problems to have such wide experience brought to their consideration. Leading from Dr. Lambe's ideas, there are several points which the writers should like to raise in discussion.

From diffuse double-layer theory, one would predict that there would be only a small influence of surface charge density on the thickness of the diffuse ion-layer or of the corresponding water layer. In the development of the theory, the second integration of the combined Poisson-Boltzmann equation requires in the boundary condition a specification of either charge or potential at the surface. In clays where the surface charge rather than the surface potential can be considered constant, this condition is usually specified from the requirement of overall electroneutrality, that the surface charge is equal and opposite to the space charge. (Theoretically it would be just as satisfactory to fix the surface potential instead of the surface charge.) Mathematically the problem is solved by setting the potential equal to infinity at some plane inside the clay surface and solving the relation between potential and distance from this plane. The distance between this plane of infinite potential and the clay surface is determined from the relationship between potential and space charge, from the conditions that the space charge must equal the surface charge. The surface charge for clay is taken from the measured cation exchange capacity. This distance turns out to be $x_0 = \frac{4}{\upsilon\Gamma\beta}$ (Schofield), where

υ = valence of cation

Γ = surface charge density

β = constant at constant temperature and dielectric constant.

The influence of charge on the diffuse ion-layer then is x_0. The maximum value encountered for the three types of clay, kalonite, illite, and montmorillonite is 4 Å for Namontmorillonite. The higher the charge density and the higher the valence of the cations, the smaller is x_0 which must be subtracted, and therefore the larger is the diffuse layer thickness. But this is only a few per cent of the total thickness in saturated clays.

a. Proc. Paper 1654, May, 1958, by T. W. Lambe.

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2. Lecturer in Civ. Eng., McGill University, Montreal, Canada.

The repulsion between clay particles may not be an electrostatic repulsion between the negatively charged units. Since only a small error is introduced by the usual assumption that the diffuse layer of cations per unit area of surface which supports it, fully neutralizes that surface charge (Childs), such a system then might have an electrical net attractive force. Langmuir has, indeed, considered that the attractive force in such a system is due to the cations. This may be more acceptable than postulating van der Waal's forces of attraction. For interaction between the relatively large clay particles there is reasonable doubt whether van der Waal's forces extend to distances of 100 Å. Therefore, a repulsion due to osmotic pressure may be the fundamental way of looking at repulsion between negatively charged clay particles. Alternatively it can be considered as an interaction of diffuse ion-layers with an excess, at the point midway between the plates, of hydrostatic pressure over the electrostatic pressure resulting from the force of the space charge.

The term double-layer is used in different ways in the literature. It was first applied to those colloidal sols which became charged by attracting a layer of ions to the surface. The second layer is composed of the ions held at some average distance from the surface. The diffuse double-layer consists then of an inner layer of ions and an outer diffuse layer. In clays where much of the charge arises within the colloidal crystal, there is really only one layer of ions, that is the diffuse one. However the nomenclature has been carried over and is now used in papers on electrochemical studies of the diffuse layer. It might be preferable not to use the term double-layer, but if it is used the layers referred to should be specified as the author has done. Any charge on the clay crystal surface due to OH^- groups corresponds more closely to the systems where "double-layer" would be appropriate.

The author has clearly defined his use of the term flocculation and dispersion. This is not questioned, but the applicability of the classical concept of flocculation may be discussed. The term was defined from experience with inorganic sols which at a fairly definite salt concentration show a clearly defined and largely irreversible flocculation or coagulation. Disturbing such a system hastens flocculation. The flocculation concentration of an As_2S_3 sol can be determined by adding various salt concentrations, and shaking after several hours. On shaking a rapid flocculation takes place above a certain salt concentration. These flocculated sols cannot be dispersed by further shaking. With clay suspensions, especially high swelling clays, there is no unique property change which defines flocculation. Shaking breaks up any flocs. One criterion for flocculation is the appearance of a clear layer of supernatant liquid under certain specified conditions, but the formation of loosely bound floc units and changes in viscosity can be demonstrated at lower salt concentration.

There need be no force of attraction involved in what is designated as "flocculation" of clay suspensions. As the force of repulsion is decreased by decreasing the thickness of the diffuse ion-layer, the particles approach each other more closely and act in unison to settle slowly. Any interaction which has been demonstrated in dilute clay suspensions is weak, and does not resemble the van der Waal's forces. There need be no unique point at which clay particles are attracted to each other and no net force of attraction. For a sol of As_2S_3 containing sufficient electrolyte for flocculation a net force of attraction exists and leads to small interparticle distances which are irreversible. Such a situation does not exist in flocculated dilute clay suspensions.

There is little evidence from studies of concentrated clay pastes that there is a point of abrupt change from repulsion between particles to attraction between particles. Rather there is a continuous transition as repulsion decreases and random orientation increases. Recent measurements (as yet unpublished) have shown that there is no discontinuity in measured swelling pressures of Na-montmorillonite in solution from 0.0001 to 0.1 N NaCl and there is no evidence of a force of attraction. The suspension in 0.1 N NaCl would be considered flocculated. There is also little difference in the water content at a given pressure of a flocculated or of a dispersed slurry during the initial compression. These data are admittedly only for Na-saturated montmorillonite. The observation that the swelling of Ca-montmorillonite is lower than the calculated from osmotic forces does not prove that a force of attraction, specifically one of van der Waal's type is present. Many of the forces holding natural clays together may be due to "cementing" substance such as iron, organic materials, carbonates or aluminum and silicate compounds.

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G. A. LEONARDS,[1] A. M. ASCE.—Potential benefits to soil mechanics and its applications of the physico-chemical approach to soil structure are virtually unlimited. The rapidity with which significant advances are being made make it difficult for the civil engineer to assimilate this new knowledge and to assess its practical utility. Accordingly, the paper by Professor Lambe, which presents these new developments with unusual clarity, is particularly welcome at this time: it is worthy of careful study by the entire profession.

An important factor not considered by the author—or, for that matter, in Soil Mechanics literature—are the properties of water itself. It has been demonstrated from X-ray diffraction and infra-red absorption measurements that liquid water molecules can exist in a variety of quasi-crystalline states; c.f. Bernal and Fowler (1933), Katzoff (1934), and others.[2] Lavergne and Drost-Hansen (1956) summarized and called attention to the discontinuities in slope (or "kinks") observed in the temperature dependence of many properties of liquid water, such as compressibility, index of refraction, specific heat, and thermal expansion. These discontinuities in slope were attributed to breakdown of water structure, which seems to occur approximately at regular temperature intervals (near 15°, 30°, 45°, and 60° C). Drost-Hansen (1956)

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2. For an excellent general discussion, see Buswell, A. M. and Rodebush, W. H. (1956). "Water." Scientific American, Vol. 194, No. 4: pp. 77-89.

showed that biological temperature optima lie between the temperatures at which breakdown in water structure occurs; for example, 37° C for mammals (half-way between 30° and 45° C), 22° to 23° C for bacteria found in soil, and 52° to 55° C for many of the thermophilic bacteria. Conversely, critical survival temperatures are near the points of discontinuities, as is the case in the pasturization of milk. Under certain conditions the structure of liquid water can be ice-like, as evidenced by observed frost damage to corn at temperatures above 40° F, and the development of a solid precipitate in natural gas lines at temperatures above 60° F. Thus, the structure of bulk water has important practical consequences; it may be expected that water structure would play an equally important role in the behavior of soil-water systems.

Phenomena observed in freezing experiments illustrate the need for considering water structure as an important element of soil structure. Dorsey (1948) found that electolytes have a negligible effect on the supercooling of water. Buehrer and Rose (1943) determined the amount of unfrozen water at -6° C in Pima Clay saturated with different cations. Their data clearly show that the percentage of unfrozen water is sensibly independent of cation concentration and valence (see Table A). These results could hardly be anticipated from the Gouy-Chapman theory. Working with compacted clay (LL = 75, PI = 50, dry unit weight 91 lb/cu. ft., water content 28%) Lovell (1957) found over 55% of the soil water remained unfrozen at a temperature of -24° C. It is the writer's opinion that a freezing point depression of this magnitude can be explained only in terms of a strongly developed water structure in the vicinity of clay particles. In this case, however, the water structure must surely be very different from that of ice to remain unfrozen at -24° C.

Striking evidence of the development and significance of water structure are obtained from diffusion experiments. Rosenqvist (1955) found that the diffusion coefficient of heavy water through clay-water systems, corrected for the presence of clay particles, was less than 10% of the value obtained by Wang (1951) for bulk water. Wang (1954) measured the self-diffusion of heavy water in aqueous solutions of electrolytes and found that certain solutes increased the diffusion coefficient. Wang attributed the increase to a breakdown of water structure. The phenomenon becomes more pronounced when the effect of varying the concentration of the electrolyte is considered (see Fig. A). As the electrolyte concentration increases, the diffusion coefficient increases, and then decreases at higher concentrations. It is the writer's opinion that at low concentrations the water structure is disrupted, thereby increasing the diffusion rate; at higher concentrations a new structure is formed, resulting in a reduction in diffusion rate. Can any other mechanism explain these phenomena?

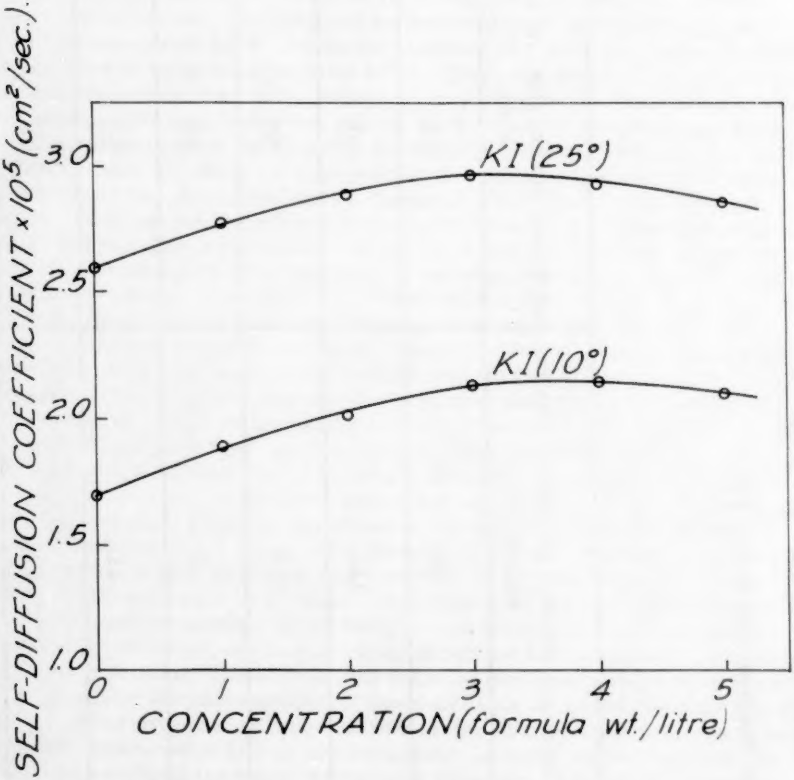
Anderson and Low (1958) showed that the density of soil water <u>decreased</u> (for a given temperature) at distances from the clay particles less than about 80 Angstroms. The smaller the distance from the clay particle surfaces, the greater the reduction in density. The high internal pressures believed to exist in the water at such small distances from the particle surfaces should tend to cause an increase in density; however, the ordered water structure apparently more than compensates for the effects of its compressibility.

Much additional evidence of the significance of water structure can be cited, c.f. Stewart (1939) and (1943), but the data presented suffice to show that the concept of water structure (orientation of water molecules) may be as important as that of particle orientation, and can hardly be neglected in the overall concept of soil structure.

TABLE A. UNFROZEN WATER IN PIMA CLAY

(after Buehrer and Rose, 1943)

Cation	Electric Potential Volts	Unfrozen Water/ gm ion of Cation Moles	Unfrozen Water/ 100 gm of Clay Moles
Ca^{++}	1.90	93	24
Cu^{++}	2.08	114	33
Mg^{++}	2.56	100	37
NH_4^{+}	0.70	42	39
K^{+}	0.75	41	36
Na^{+}	1.02	49	38



VARIATION OF THE SELF-DIFFUSSION OF WATER IN SOLUTIONS OF POTASSIUM-IODIDE WITH CONCENTRATION AND TEMPERATURE

FIG. A

(After Wang, 1954)

The author's explanations of such phenomena as strength gain with time, creep, secondary compression, and loss of strength past the peak strength are considered as simplifications intended to illustrate the generality of the physico-chemical approach. The writer is of the opinion that these phenomena are more complex than the author's explanations suggest.

The author has listed reduction in temperature among the factors contributing to reduced shear strength in clays. The writer is aware of only three records on the relationship between temperature and shearing resistance of clays; Hogentogler and Willis (1936), Rosenqvist (1955), and Trask and Close (1958); all three report an increase in strength with decrease in temperature. Fig. A shows that a reduction in temperature reduces the self-diffusion coefficient significantly, which is indicative of a more strongly developed water structure and probably greater shear strength. The author's comments would be appreciated.

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THE ENGINEERING BEHAVIOR OF COMPACTED CLAY[a]

Discussions by Alfred C. Scheer, R. N. Y. Yong and B. P. Warkentin

ALFRED C. SCHEER,[1] J. M. ASCE.—The excellence of Mr. Lambe's paper is self-evident. It deserves much praise. However, one factor which might be worthy of some further consideration, in connection with permeability, is the non-isotropic character of compacted clay.

The treatment of permeability in the early part of the paper is written as though compacted clay specimens were isotropic, permeability-wise. No mention is made of the direction of permeability measurements. Were the permeabilities reported in Fig. 1 and Fig. 2 measured horizontally or vertically? Also, the formulas on page 2 are presented without any mention of the direction of flow. Both K and k_0 would be different for different directions of flow.

The extent and nature of the differences in permeability in different directions would seem to be a matter of considerable importance in studies of this type, especially in cases where the clay platelets tend to be oriented with their faces parallel.

R. N. Y. YONG,[2] J. M. ASCE and B. P. WARKENTIN.[3]—The paper by Dr. Lambe presents a keen analysis of his studies of the mechanical and physico-chemical behavior of compacted clay. This is a most valuable contribution to this particular phase of basic research.

The author discusses the influence of temperature on electric potential, and shows in Fig. 2 of his paper that the diffuse ion-layer expands or increases in thickness with an increase in temperature. The thickness of this layer is usually specified as the distance over which the potential drops to some fraction of the potential at the surface ($\frac{1}{e}$ if $\frac{1}{k}$ is considered the diffuse layer thickness, as usual by Kruyt). The compaction results, however, show that the clay compressed with an increase and expanded with a decrease in temperature. This appears to be opposite to that predicted theory, which considers expansion due to an increase in thickness of the diffuse ion-layer.

Gray's results on the consolidation of fine-grained soil, wherein an increase in temperature resulted in a steeper secondary compression curve, seem to indicate that more water is extruded from the contact areas at a

a. Proc. Paper 1655, May, 1958, by T. William Lambe.
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higher temperature. If the concept of three types of water, i.e. bound water, double layer water and free water is adhered to, this would seem to be at odds with the theory showing that the double layer increases with temperature—thus resulting in less free water. This assumes that the water extruded is free water. If free water is not available for the expansion of the diffuse ion-layer such that the original space of the free water is taken up by expansion, then any expansion of the diffuse ion-layer would cause a resultant expansion in the soil mass. However, considering available free water, the greater the expansion of the diffuse ion-layer, the less would be the amount available free water. However, as stated, this expansion of the diffuse ion layer tends to decrease the soil strength which may offset the effects of the reduction of the available free water. It is also stated that soil strength, which is decreased by an expansion of the diffuse ion-layer, decreases with a decrease in temperature. From theoretical reasoning, with a concentration of the diffuse ion-layer, as a result of temperature decrease, the soil strength should then be increased.

If a net force of attraction or of repulsion between particles were present, and if this net force, being the resultant of forces of attraction and of repulsion, could be separated into these individual forces of attraction and of repulsion, then they could be represented schematically in the following manner. (See Fig. A) This schematic diagram appears to indicate that without any external force an equilibrium condition could be reached whereby the attractive forces would balance the forces of repulsion. This equilibrium condition would then establish an equilibrium distance between parallel particles. Any separation other than this equilibrium distance would result in a net force of attraction or of repulsion as the case may be. It is generally agreed that since only net forces can be measured it may not be correct to indicate a separate curve for the force of repulsion and another for the force of attraction, and that a more accurate presentation would be a graph showing only a net force. (See Fig. B)

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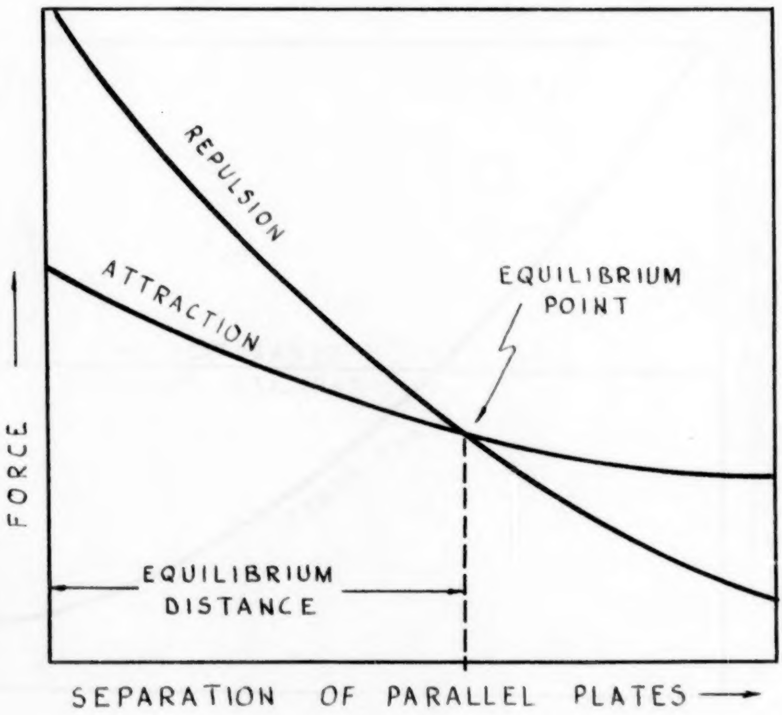

Fig. A - Schematic Diagram representing Force variation with Separation of Parallel Plates

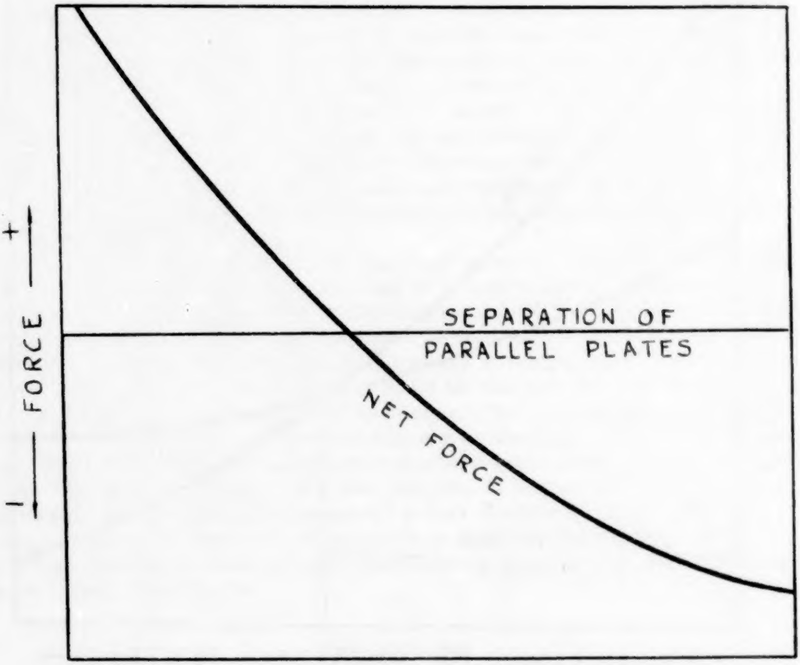

Fig. B - Schematic Diagram representing Net Force with Separation of Parallel Plates

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

VOLUME 83 (1957)

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

VOLUME 84 (1958)

JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

c. Discussion of several papers, grouped by divisions.

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